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IRRIGATING IN THE HUMID AREAS[1]

E. A. Kimbrough, Jr.[2]
(Proc. Paper 1352)

SYNOPSIS

A comparison is made of the problems of irrigating in the humid with those of the arid area. Predicting the date of occurrence of a serious drought is a major problem in the humid area. Developing streams flow for irrigation involves drainage requirements. Crop response, being the measure of success, varies considerably with date of planting and date of the drought. Labor requirements are a function of design field size, and cropping system.

This paper is designed to report to this group assembled on the status and problems encountered when irrigation is considered as a part of the agricultural practices used in crop production in the humid areas of the United States. An attempt has been made to explain the differences between the problems of the arid areas and those of the humid area, especially the South.

Irrigation in the South or humid area as it is today is not a new concept of agricultural production. In Mississippi, supplemental irrigation was tried on several occasions, and all proved to be failures. There is a report that in the early 1930's, a farmer in the Mississippi delta attempted to irrigate from a small lake. This irrigation was not profitable, but the condition created in the lake made seining for "spoon bill" catfish very profitable for him. This is an example to show that irrigation was attempted approximately 20 years before the present era.

The problems of irrigating in the humid area may be classified into the following groups:

1. Climatological
2. Water resources and development

Note: Discussion open until February 1, 1958. Paper 1352 is part of the copyrighted Journal of the Irrigation and Drainage Division of the American Society of Civil Engineers, Vol. 83, No. IR 2, September, 1957.

1. Presented at a meeting of the ASCE in Jackson, Miss., February 19-22, 1957.
2. E. A. Kimbrough, Jr., Asst. Agri. Engr., Mississippi Agri. Experiment Station, State College, Miss.

3. Agronomic
4. Engineering design
5. Economics

In any business, as well as agriculture, unless an operation has an economic gain, it shall soon be rejected as a failure. Therefore, irrigation must show a net gain with sufficient margin for planters and farmers to exhibit any great interest. The economic factor will not be discussed in the material, except to demonstrate some point in the other classifications of problems.

The first studies of the present irrigation era were begun in 1946 by John R. Carreker and W. J. Liddell at the University of Georgia. Due to increased intensity of the drought years that followed these initial investigations, studies are now conducted by all the state experiment stations of the humid areas.

Carreker and Liddell(3) reported on drought frequency based on the definition of a drought being a period of time of 14 days or longer in which less than 0.25 inch of rainfall occurred during any 24 hour period. This study indicated that the two week duration was of such frequency that crop production could possibly be increased by supplemental irrigation. The drought period that would possibly affect the most common crops occurred from June to September. However, the crops produced in the autumn are likewise affected, especially the winter forage crops. A comparison of the drought periods for Mississippi and Georgia are shown in Figure 1-a and 1-b, respectively. The occurrences of the drought periods for these locations are similar, but they are not the same as to date and actual length. Therefore, a climatological problem in the humid area is predicting with accuracy the date of occurrence of a serious drought and the effective length of the drought.

The effective length of the drought is the period between rains of adequate quantity and proper rate. Rains which exceed the intake rate of the soil are not necessarily effective in ending a drought. The rainfall rate above the intake rate of the soil becomes runoff instead of soil moisture. On numerous occasions, soil has been found wet only 3 to 4 inches deep, and the rainfall was one inch or above. The ditches and drainage canals were full from the runoff. These thunder storms and general rains may also be very damaging to crops if irrigation has been applied earlier. Where irrigation is to be used, adequate drainage is a prime requirement to decrease the hazard of drowning a crop.

As an example, the rainfall of which by definition is sufficient to break a drought, occurred twice in September and once in October, 1953, at State College, Mississippi, but the rainfall was not beneficial to the fall crops. The actual length of the drought was 95 days, the summation of 4 periods which by definition were individual periods.

When farmers are to use irrigation, the schedule on which it is designed must be followed. For any system to be economical, the frequency or period allowed to cover the design area will be approximately a minimum of seven days on most crops and soils and as high as fourteen days or more on certain soil types. If the weather forecast is "scattered showers" in this farmer's area, the question becomes, "Should he irrigate or see what the weather is going to do?" If his crop is at a critical stage of growth, a delay could be very costly because local showers do not follow a set pattern as where they will fall. If the irrigating farmer delays his irrigation and the local showers miss him, the area that will suffer is that portion of his crop which is on the

end of irrigation period. There is a tendency of the Southern farmer to wait too long to begin his initial irrigating and even allowing his complete crop to suffer when irrigating facilities are available. Gambling on the weather is the cause of these failures, and the problem is predicting accurately the location and date of rainfall in the humid area.

The problem of water supplies in the humid area is becoming a complex and serious matter. During the recent drought years, many streams that were at one time thought to be perennial have become intermittent streams. The seventeen western states use the melting snow from the mountains as a part of their total water supply. The humid area does not have this natural storage facility as the precipitation is in the form of rain. Natural drainageways become filled during the spring and winter, moderate flow in early summer and some become dry in late summer and fall. Streams which are remaining perennial are often taxed to their limit already for supplying irrigation water. An interesting stream located in the Mississippi Delta is the Bogue Phalia.(4) Rice producers and cotton farmers have pumped the stream to its limit. The calculated area it could safely irrigate from 1952 was 3990 acres. Other farmers along the bank of this stream must rely on ground water or take a change that stream flow would increase or the other farmers do not need the water at this time and the water he takes out would be lost anyway. Some of these problems may be solved as more water resource legislation is passed, but it is a major problem at the present time. Also, these streams are major drainageways and there should not be interference with this function of these streams.

The ground water supply in the Delta area of Mississippi has been predicted adequate for future agricultural and industrial development expected within the general area, but other areas of the southeast are not as fortunate. Wells in the Delta area may be developed to produce up to 3000 gpm from a depth of only 100 to 200 feet. Static heads are as low as 16 feet in some cases. This is the most reliable source of irrigation water in the area. The initial investment is normally low enough to warrant this type of water supply instead of using a stream which may not be reliable as well as increasing the transportation cost of the water to the fields.

To illustrate the problem of ground water development in an area near Holly Springs, Mississippi, wells vary from 250 feet to 400 feet of depth. A 150 gpm well installed at the North Mississippi Branch Experiment Station cost $6,000 and this price did not include the power unit which would have cost the station approximately $850. An area in northeast Alabama is even worse than this in that there is not anything but solid rock beneath the surface. Areas similar to these must reply on surface retention or adequate stream flow which may be regulated by surface retention.

If surface retention of water is to be used, the problems in the southeast may be listed as:

1. Site adapted to the storage of water
2. Protection of the dam from overtopping
3. Type soil present of site to make reservoir water tight
4. Siltation
5. Predicting the quantity of water available for irrigation in the reservoir

Having a good site for storage of water does not necessarily indicate it advisable for irrigation. Numerous sites have been investigated in Mississippi as well as in other southern states and have been eliminated as possible

irrigation reservoir sites because the land around it could not warrant the use of irrigation for the lack of a feasible crop.

A surface reservoir may be designed to have only the total expected watershed yield to run into the reservoir and not have any need for the spillway except when the watershed yields are above the design runoff. This type design procedure is assuming that the water to be used or loss through evaporation each year is equal to the runoff during the winter months. The fallacy of this theory is the water may not be needed due to ideal or close to ideal showers of rain and, therefore, leaves the surface of the water at a higher elevation than the design allowed. The expected watershed yields will cause spillway elevation to be reached earlier and expose the dam to the heavy rain that follows. Therefore, in the Southeast the reservoir design must be based on the theory that the heaviest of rain will occur after the reservoir has reached spillway elevation and provide adequate spillway capacity for this condition.

Several areas in South Carolina and Mississippi have coarse sand at these sites which would cause deep seepage of the stored water. Investigations are being conducted on methods of preventing these losses by A. W. Snell,(5) but most of these methods are still rather expensive. The two methods which offer the most in this problem are the use of plastic sealers and dolomite clay.

Another problem of developing irrigation water by this method is the property boundary. A natural site for farm "A" backs the water up on the land of farm "B." Farmer "B" objects to this water as it is of no benefit to him, but the irrigation potential has been decreased due to this situation.

Many areas are highly erodible and will cause excessive siltation of the reservoir. These areas should be avoided as the investment would be on a short term basis.

The greatest problem in the Southeast is predicting the available water supply stored. The total quantity stored is not available for irrigation as losses will be encountered which are:

1. Evaporation daily from the surface of the water in the reservoir
2. Seepage loss
3. Volume to remain in the permanent pool

Since the reservoir area will be the greatest when the water level is at spillway elevation, the daily losses by evaporation in total quantity will be large. A volume depth curve is shown in Figure 2 to illustrate the problem.

Volume "L," the loss by evaporation and seepage is a function of the time that expires between the full pond at point "A" and the date of irrigation is begun at point "B." The volume "I" remaining in the reservoir at "B" may be needed for irrigation for a short drought of two weeks or it may be needed for a drought that will extend beyond thirty days with some permanent pool reserved for livestock use. The volume at "C" should not be pumped from the reservoir. The basic method to employ at the present time is to assume the drought expected shall equal or exceed any drought that has been severe enough to harm the crop on which the water will be used as reported by those doing research on crop response. This will not be the most economical method to use, but there will not be a shortage and no danger of overextending the water supply. Also, the only benefit of summer showers is to assist the crop and to counteract evaporation and seepage losses. Very little runoff will occur during the summer months.

Furrow irrigation is not as efficient as sprinkler irrigation from the water use standpoint; however, there are several factors which should be considered in the southeast. These are:

1. Small fields in the hill section
2. Soil variability making certain fields better adapted to certain crops.
3. Tall field corn which must have a very low cost of irrigation
4. Drainage
5. Small total farm size

The southeastern section of the United States is predominantly an area of small farms. There are, of course, sections similar to the Mississippi delta where large land owners operate but they are in minority. Average size farms for various states in the humid area and the number of farms with 200 acres or larger cropland are shown in Table 1.

These small farms are further reduced to small fields. Each field is considered good corn land, pasture land, cotton land, and garden or truck patch land. When irrigation is considered, the system used may be sufficiently large to irrigate a field of corn in two days. The next field may be one-half mile away which will require that the complete system be moved—both sprinkler and furrow. This reduces the efficiency of the system. In time requirement studies made at State College, Mississippi, 10 man hours of labor were required to irrigate an acre of corn by sprinkler as compared to 3.5 man hours by furrow. The main difference is the time required in laying the lateral pipe and placing the riser tripods and risers on the lateral. Moving the complete system to the next field is the reason for the high man hours for furrow irrigation in this area. Therefore, the design capacity of an irrigation system in the southeast cannot be based on the total number of acres alone, but sufficient time must be allowed for moving the system from place to place on the farm. The net result is a larger investment becomes necessary as compared to extensive irrigated fields of the western states.

A new system of sprinkler irrigation has been tested at Alabama Polytechnic Institute by Bouwer and Helms.(6) This system is known as the branched lateral design in which the nozzles are not connected directly to the lateral pipe but to a plastic hose. The hose length is equal to the spacing between lateral lines in a conventional system. In essence, three settings of the lateral are accomplished by moving the nozzle only instead of the complete line. Labor requirements reported averaged one-man hour per acre. This system may have a very wide application in the South.

In the Delta area, due to the natural flat terrain, farmers are becoming more interested in "land forming" and the use of gravity or furrow irrigation. The reasons for the shift from sprinkler to furrow irrigation are:

1. High power requirement for sprinkler
2. Total investment for sprinkler equipment as compared to that of furrow irrigation
3. Labor requirements for sprinkler were excessive
4. "Land forming" has other benefits in their farming operation, mainly drainage and increased efficiency of machinery for mechanization
5. Large volumes of water may be managed by surface channels with less labor than with sprinkler

The main factor that will determine the success of irrigation in any area is the results obtained by crop response. The crops produced in the humid

area primarily are cotton, corn, soy-beans, tobacco, and forage crops such as clover, sorghum and grass pastures. There are a few areas in which vegetables are produced. The results from the Delta Branch Experiment Station located at Stoneville, Mississippi, are shown in Table 2 for cotton and corn as conducted by Perrin H. Grissom.(7)

A test on date of planting conducted by Jordan, Ratliff, and Kimbrough on corn at Mississippi State College indicates that the date of planting affects the results obtained by irrigation. Results are shown in Table 3.

In analyzing these data, yields are not consistent with the irrigation water applied. Corn yields are more consistent in giving a response to irrigation, but if the early planting date is used, irrigation requirements are reduced or the available water supply may be extended. Cotton response varies with the rainfall received and the effective rain may occur within a few days after beginning to irrigate. One other great problem involved in the southeast in cotton is the control of insects, and irrigation makes the problem more difficult because of the sequence of irrigation versus sequence recommended for insecticide application. Associated with this problem is recommended rates of fertilizer used. During season of excessive rainfall, the vegetative growth produces a stalk too rank for mechanized machinery, especially if the nitrogen fertilizer is applied heavy. During the season where irrigation is needed, these heavy rates do exceedingly well in response. The farmers in the southeast must use irrigation as an insurance policy. Therefore, to be safe in the overall program, modification of the fertilization program must be made because the weather of an individual year cannot as yet be predicted.

An attempt has been made to discuss some of the major problems of irrigation in the Southeast as observed in research studies at State College and reported from other humid areas. Irrigation is apparently going to remain in the humid areas but the future should not be based on a single year in which very little rainfall was received or where the opposite occurred as in 1955. The type system used will depend on the physical properties of the individual farm, the operator's likes and dislikes, and more important the crops he produces.

SUMMARY

1. Drought periods do not occur on the same day or week of a given month from year to year making it difficult to predict with accuracy the date irrigation is to be used in the humid area.

2. Irrigating a crop which is followed by rain can reduce yields, especially without proper drainage.

3. Water supplies are becoming an important item as irrigation expands. Major streams are drainageways and nothing should interfere with this function.

4. Small farms and small field sizes cause a high investment per unit of land irrigated by sprinkler system.

5. Production practices are influenced by irrigation and these practices and irrigation must become companions for the success of either.

6. "Land forming" benefits are not limited to irrigation alone. Drainage and mechanization are improved by this practice.

7. Farmers can benefit from early crops by reducing the hazard of drought which occurs more frequently during the summer and early fall.

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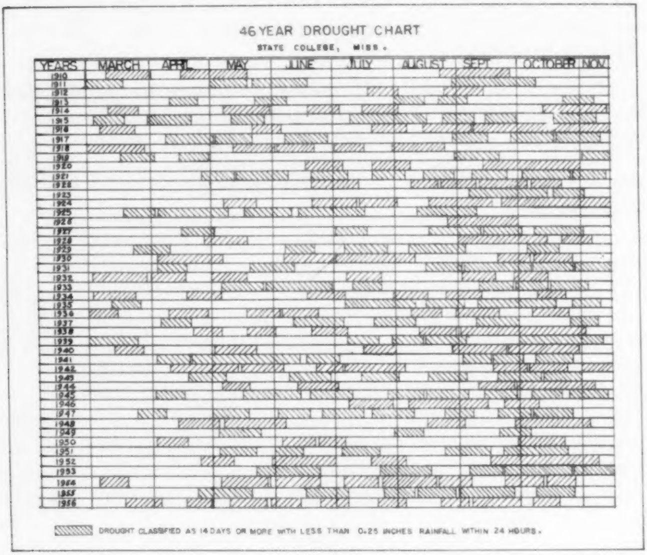

FIGURE 1-a: Drought periods at State College, Mississippi 1910....1956

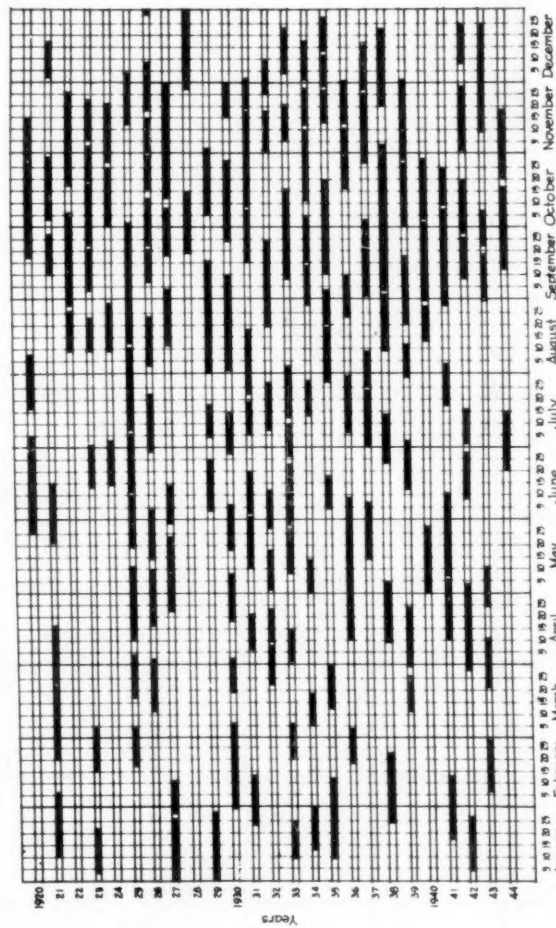

FIGURE 1-b: Drought periods at Athens, Georgia 1920....1944

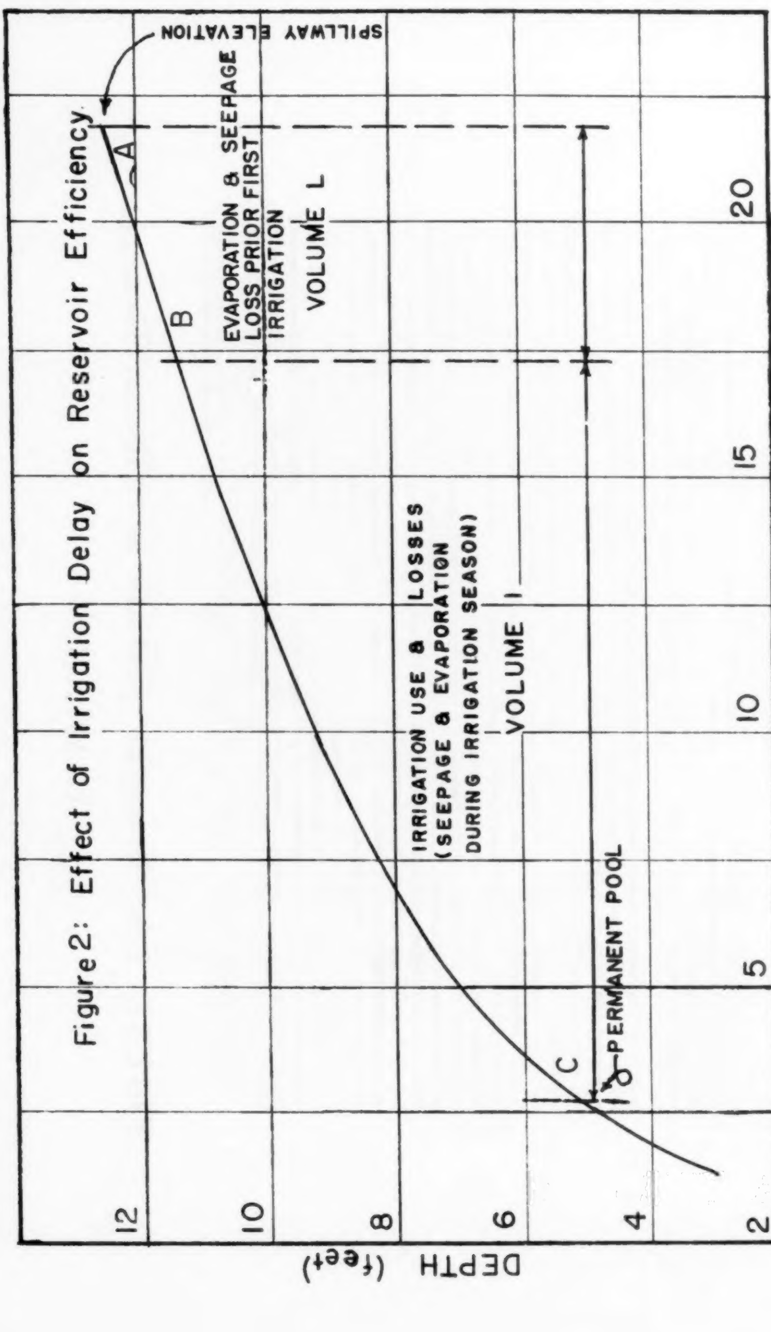

Figure 2: Effect of Irrigation Delay on Reservoir Efficiency

Table 1: Farm Sizes in the Southern States*

State	No. of Farms	Average Size Acres	No. of farms with over 200 acres cropland
Alabama	176,956	117.6	1,840
Arkansas	145,075	123.7	4,978
Georgia	165,524	145.1	3,575
Louisiana	111,127	103.0	2,605
Mississippi	215,915	95.9	2,983
N. Carolina	267,906	68.2	982
S. Carolina	124,203	89.1	1,672
Tennessee	203,149	86.9	1,475

* 1954 Census of Agriculture, U. S. Department of Commerce

Table 2: Yields and Irrigation Requirements for Cotton and Corn*
Stoneville, Miss.

Year	Corn: Times irrigation applied	Corn: Yields Bu./Ac.	Cotton: Times irrigation applied	Cotton: Yields - pounds seed cotton/acre
1953	0	69.5	0	3001
	1	70.5	1	3542
	3	86.7	3	3522
	5	93.0	5	3724
1954	0	73.0	0	2443
	1	77.1	2	2959
	3	94.9	5	3437
1955 1/	0	86.7	0	2657
	2	111.8	1	2695
1956 2/ 3/	0	73.4	0	1502
	1	88.2	1	1531
	2	102.3	4	2686
	3	102.3	6	2884

* Annual Reports, Mississippi Agricultural Experiment Station

1/ Irrigation not required until very late in the season because of the general rains received. Cotton received 150 lbs. Nitrogen per acre. Corn received 180 lbs. Nitrogen per acre for data shown.

2/ Cotton was first irrigated after 3rd week of bloom. Corn tests irrigated at first tassel (June 28).

3/ 1956 data presented by Perrin Grissom at the Mississippi Agricultural Experiment Station Annual Meeting, State College, Mississippi, December, 1956.

Table 3: Results of Irrigation vs. Planting Date
State College, Miss.
1956

Date of planting	Variety	Yield of Irrigated (bu./ac.)	Yield of Non-Irrigated (bu./ac.)	Water Applied by Irrigation (Inches)	Rainfall to 120 days after planting (Inches)
April	Dixie 33	102.6	106.5	4.6	11.42
	Dixie 18	78.1	70.4	4.6	11.42
May 14	Dixie 33	117.0	96.2	4.6	10.10
	Dixie 18	103.8	81.0	4.6	10.10
June 18	Dixie 33	79.8	60.2	11.4	7.84
	Dixie 18	65.9	33.9	11.4	7.84

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SALT BALANCE IN GROUND WATER RESERVOIR OPERATION

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(Proc. Paper 1359)

SYNOPSIS

Planning by the California Department of Water Resources for development of water in California to meet the State's ultimate water requirements has revealed the necessity of planned operation of the State's ground water reservoirs for seasonal and cyclic storage of water. A problem that must be solved in such operation is maintenance of suitable mineral quality of the ground water. This paper discusses the sources and disposal of salts in ground water reservoirs, presents an illustration of relationships and requirements for water to maintain salt balance in a hypothetical ground water reservoir, and discusses the deficiencies in present knowledge and in data collection programs for evaluation of salt balance problems.

INTRODUCTION

Development of California's water resources to meet ultimate water requirements will require the use of all economically feasible surface reservoirs and, in addition, planned utilization of the immense storage capacity of the State's many ground water reservoirs. The gross usable storage capacity of the more important of these basins is estimated to exceed 130 million acre-feet. This capacity is usually estimated for the interval from 20 to 200 feet below ground surface, although in some basins now dependent largely or entirely upon ground water, pumping from considerably greater depths has been found economical. Utilization of this storage capacity requires planned operation of the basins(1,2,3) under which the water level is drawn down during dry periods thereby making available storage capacity to retain a portion

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Note: Discussion open until February 1, 1958. Paper 1359 is part of the copyrighted Journal of the Irrigation and Drainage Division of the American Society of Civil Engineers, Vol. 83, No. IR 2, September, 1957.

1. Superv. Hydr. Engr., Calif. State Dept. of Water Resources. Los Angeles, Calif.
2. Superv. Hydr. Engr., Calif. State Dept. of Water Resources. Los Angeles, Calif.

of local runoff and/or imported water during wet years.

One of the important problems that must be considered in effective utilization of ground water storage is maintenance of favorable salt balance. Salt balance has been defined by Scofield(4) as ". . . the relationship of salt input to salt output." He amplified this by stating: "If the mass of the salt input exceeds the mass of the salt output the salt balance is regarded as adverse, because this trend is in the direction of the accumulation of salt in the area and such a trend is manifestly undesirable." Considerable attention has been given to salt balance in river systems where diversion and irrigation with surface water is practiced and to salt balance as it affects accumulation of salts within the root zone of plants. Much less attention has been directed to the equally important problem of maintenance of favorable salt balance in those areas where a ground water reservoir is the primary source of supply as well as the facility for seasonal and cyclic storage of water. Primary consideration is given herein to the economic significance of salt balance. Salt balance is discussed both for basins used to store and regulate imported water, and for basins where the entire water supply is derived from precipitation within the boundaries of the tributary watershed. No consideration is given to the many complex legal considerations or to the physical problems other than withdrawal of water that must also be solved to accomplish effective planned utilization of the storage capacities of these ground water basins.

Source of Salts

The major quantities of salts are usually water-borne, but the quantities imported to an area by man in other forms cannot be ignored.

The natural sources of salt are: (1) salt content of local surface streams tributary to the ground water basin; (2) salt contained in subsurface inflow to the ground water basin; (3) salt leached from the soil above the water table by deep percolation of water; and (4) salts leached from aquifers below the water table.

Sources of salt resulting from man's activities include: (1) the salt load of imported water; (2) salt leached from surface soil during each cycle of use and return percolation of ground water; (3) soluble minerals derived from fertilizers, soil conditioners, insecticides and other agricultural chemicals, and carried down by percolating water; and (4) the increment (estimated as 100 to 300 parts per million (ppm) total dissolved solids)(5) added by domestic and industrial wastes of various kinds. Seawater intrusion in coastal ground water basins falls into the above category; however, it is not considered herein since its occurrence so far overshadows all other effects that it must be considered a distinct problem. The same consideration applies to those basins where connate brines underlie fresh water at shallow depths, and are disturbed and mix with the fresh water because of heavy pumping draft.

Concentration Salts

Repeated cycles of use of water for most purposes results in evaporation of a portion of the water without a corresponding reduction in the total quantity of salt. This concentration, whether due to irrigation, domestic and municipal, or industrial uses, can result in salt content above desirable

limits. Evapo-transpiration in high ground water areas may result in substantial increase in concentration of salts.

The concentration of salts resulting from cycles of use and re-use can readily be illustrated with data obtained for a number of western streams. For example, Haney and Bendixen(6) used United States Bureau of Reclamation data for the Arkansas River in the year 1940-41, to show an almost sixfold increase in concentration of salts from 346 to 2034 ppm, in a 176-mile reach of the stream. In the same reach of the stream 334,200 acre-feet of water plus all tributary inflow were consumptively used.

There is evidence of similar concentration of salts in ground water basins, although, due to slow movement of the water and large storage capacity, it is much more difficult to illustrate.

In the Upper Santa Ana Valley of southern California, surface inflows which largely replenish the ground water basins average 100 to 150 ppm in dissolved salts. Rising water outflow from the series of ground water basins in this valley had an average salt concentration of 450 to 500 ppm in 1907-08 before any appreciable effect of man's occupation of the land. By 1955 the salt concentration in the rising water outflow had increased to values ranging from 600 to 650 ppm.

Disposal of Salt

It follows from the definition of salt balance, that the salt removed from a ground water basin must equal that which comes in from all sources if the basin is in balance. If an adverse condition exists, i.e., more salt entering than leaving the basin, the salt concentration in the ground water and in the outflow will increase until a balance is effected. The basic problem is to provide sufficient water to bring the basin into balance before the ground water becomes too saline for use. The principal means of disposal of salts are: (1) salts in surface and subsurface outflow, (2) salts contained in exported water, and (3) salts contained in exported sewage and industrial wastes. The quantity of salts contained in agricultural and manufactured products shipped out of the area is usually negligible in comparison to the quantity in the other categories.

There are some ground water reservoirs in which there is no subsurface outflow and from which export of water or wastes may not occur. Periodic maintenance of ground water levels high enough to obtain the effluent seepage required for salt balance may result in water logging extensive portions of the overlying lands. The only apparent alternative would be pumping of sufficient water from the lower portion of the reservoir. Unless a substantial portion of the cost of such pumping can be justified for drainage, it may not be economical to operate a reservoir involving such an unfortunate combination of circumstances.

Allowable Salt Concentrations

A further consideration is the degree of salt concentration that can be tolerated without undue impairment of beneficial uses of the water.

Domestic and Municipal Supplies

The United States Public Health Service Drinking Water Standards(7) are

applied in many areas as the criteria of suitability of water for domestic and municipal purposes. Specific allowable concentrations of dissolved minerals are given in Table I.

Irrigation Supplies

Quality requirements for irrigated crops are in some respects more severe than those for drinking water. Quality criteria widely used in California for irrigation water[8] were formulated by Dr. L. D. Doneen of the University of California, College of Agriculture at Davis. This approach recognizes three broad classes of water as defined by four parameters, shown in Table II.

These classifications are necessarily arbitrary. Suitability of a water for irrigation is also dependent upon such factors as type of crop, soil type and structure, climate, and rainfall distribution. Dr. Doneen has recently developed a newer approach to evaluation of suitability of irrigation water which he has termed "effective salinity."[9] This method considers the proportion of the salt in the water than can be expected to precipitate from the soil solution. The effective salinity is then the measure of soluble salts which will tend to increase in concentration in the soil solution. This criterion is gaining increasing acceptance in California, and is worthy of consideration in water quality studies.

Industrial Water Uses

Quality requirements for industrial purposes vary widely according to the type of industry and also for the diverse uses within a particular industry. For some purposes, such as once-through cooling, or log ponding and booming, water of quite inferior properties might be acceptable. Whereas, for use in laundering, ice manufacture, boiler feed, food processing and beverage industries, the quality requirements are stringent. In general, highly mineralized waters would be unsatisfactory without elaborate treatment for any but rough industrial uses.

Typical Salt-Balance Studies

Salt-balance problems that have been discussed in the literature fall into two categories. The first are those studies of river basins where the consideration is the quantity of salt that enters an irrigated valley situated on a major river as compared to the quantity carried out at the lower end of the valley in the river water. The published studies concern areas where irrigation is accomplished mainly by diversion from the streams without much utilization of ground water storage. A second common type of study has concerned salt balance in the root zone of crops. These latter studies are designed primarily to determine the irrigation and drainage practices necessary to prevent undesirable buildup of salts in the top few feet of soil. Again, ground water is either not used, or little consideration has been given to the effect on its quality when salts are carried downward into underlying aquifers. Detailed water quality studies of ground water basins have dealt largely with sea-water intrusion, lateral or upward diffusion of connate brines, or uncontrolled industrial waste discharges. It is doubtful whether sufficiently careful measurement and inventory of the salts entering and leaving any basin have been made to permit an accurate evaluation of the slower and less

spectacular but important trends toward adverse salt balance. The following study is largely theoretical and will serve to illustrate rather than exemplify this type of problem.

Salt-Balance Requirements for Operation of a Reservoir With Local Water Supplies

To simplify the illustration, only salts derived from surface and subsurface inflow are considered. It is assumed that all salts leaving the reservoir, whether in effluent seepage or pumping extractions at the lower end of the reservoir, enter a surface channel where the quantity of water can be measured and the salt content determined.

The ground water reservoir is assumed to have an area of 75,000 acres, an average depth of alluvium of 320 feet, a specific yield of eight per cent, a saturated depth of 300 feet, and the water in storage contains an average of 500 ppm total dissolved salts. There is no subsurface outflow and effluent seepage can be regulated by drawing down the water level in the reservoir. There is sufficient surface storage available to prevent uncontrolled outflow of flood flows.

With the ground water reservoir operated with local water supply only, the average annual supply and disposal of water are as shown in Table III.

It is assumed that re-use of ground water can be controlled to permit outflow at concentrations of either 1,000 or 1,500 ppm. To prevent excessive salt buildup in the reservoir there must be an average annual outflow of 37,000 acre-feet at 1,000 ppm, leaving a net usable supply of 75,000 - 37,000 = 38,000 acre-feet of ground water. With a concentration of 1,500 ppm the required average annual outflow is 24,700 acre-feet and the net usable ground water supply is 50,300 acre-feet. A net annual increase in ground water supply of 12,300 acre-feet is realized when the concentration of the outflow is raised from 1,000 to 1,500 ppm. It is reiterated that, for the sake of simplicity and because of lack of information, this illustration has not considered certain important sources of salt.

The importance of the yield of the ground water reservoir can be illustrated in terms of land supplied with water and value of the water. If the gross consumptive use of water in the area is three acre-feet per acre, of which one acre-foot is supplied by rainfall, the reservoir yield will supply 19,000 or 25,150 acres depending on the concentration of the outflow and the corresponding amount of water used to maintain salt balance. The cost of water in California ranges from less than one to more than forty dollars per acre-foot. If a cost of ten dollars per acre-foot is assumed in this case, the value of the water obtained from the reservoir will be either $380,000 or $503,000 annually, depending upon salt-balance requirements.

Salt-Balance Requirements for Operation of a Reservoir with Local and Imported Supplies

This same hypothetical ground water reservoir is used to illustrate the effect of salt-balance requirements on importation of supplemental water. The local supply is sufficient to furnish water for 39,000 acres plus an average depth of one foot of water from precipitation for the remaining area when the concentration of salt in the surface outflow is 1,000 ppm. If 90 per cent of the area is furnished its total water requirement each year, an additional 57,000 acre-feet annually will be required to meet the consumptive-use requirements. However, if this supplemental water contains 400 ppm total

dissolved solids, it will be necessary to import an additional 38,000 acre-feet of water for use in removing the salt from the basin at 1,000 ppm concentration. The additional water required for salt balance would cost $380,000 annually at $10 per acre-foot or the net cost of the imported water consumptively used would be increased from $10 to $16.67 per acre-foot. If, as in the example of operation with local water supply, a salt concentration of 1500 ppm could be tolerated, 44,700 acre-feet of supplemental water would be required for consumptive use. The additional water required to remove the salt would be reduced to 16,250 acre-feet annually and the net cost of the imported water consumptively used would be decreased to $13.64 per acre-foot.

Use of Ground Water From the Reservoir During a Drought

The effect of drought on average quality of water in the ground water reservoir, assuming complete mixing in the reservoir, can be computed. A period of four dry years is used for illustrative purposes. It is also assumed that precipitation was sufficient to supply one foot of consumptive-use requirements annually with no excess available for recharge, no ground water inflow occurred, surface inflow and importation were reduced to 20,000 and 28,500 acre-feet per year, respectively, and effluent seepage was prevented by the lowering of the water table. With the area, depth, and specific yield previously mentioned, the reservoir would have a storage capacity of 1,800,000 acre-feet.

Total salt in storage at the beginning of the period, if the average concentration were 500 ppm, would be 1,224,000 tons and salt added during the period would be 106,000 tons or a total content at the end of the period of 1,330,000 tons. Withdrawal from storage for consumptive use of 346,000 acre-feet of water would result in a concentration of

$$1{,}330{,}000 \div (1{,}800{,}000 - 346{,}000)\,(0.00136) = 673\text{ppm}$$

This suggests that substantial use can be made of storage capacity in a reservoir without unreasonable deterioration in quality of the water therein.

Deficiencies in Present Knowledge of Salt-Balance Factors

The necessity of creating the foregoing hypothetical example of salt-balance computations rather than using an actual basin demonstrates the limited accomplishments to date in operating ground water reservoirs as well as the many limitations in our knowledge of the quantitative values of the factors of salt balance involved. There are few reservoirs in which both the areal and vertical ground water quality is sufficiently well known to permit computation of average quality and even fewer where such data have been collected for a sufficiently long period to permit accurate correlation of change in quality with related hydrologic data. Much more precise data must be obtained for quality of water entering and leaving the reservoir than are generally available at present. Perhaps the greatest deficiency, however, is in our knowledge of the quantities of salts added to a basin in the form of fertilizers, soil conditioners, weedicides and insecticides, and by man's other activities. Little is known about salts leached by the percolation of water downward from the surface. Evaluation of data now available has also demonstrated that there are appreciable time lags between inflow or outflow of salt and proportionate changes in concentration of salt in stored ground water. The reasons for such time lags are not yet apparent.

Magnitude of Salt-Balance Problems in California

Studies of water requirements for The California Water Plan show that some 25,800,000 acre-feet of supplemental water will be required to meet ultimate consumptive requirements in the portion of the State south of the latitude of Sacramento. Most of this water will be obtained from the watersheds to the north. The additional quantity of water required to maintain a favorable salt balance will be a major consideration in determination of size of facilities required to regulate and transport the supplemental water since much of it will be used on lands overlying ground water reservoirs and the storage capacity therein must be utilized for seasonal and cyclic regulation. If ten per cent of gross deliveries at a cost of $10 per acre-foot are required to maintain salt balance, the value of the water will be of the order of $25,000,000 annually and justifies the studies necessary to achieve reasonable economy in its use.

CONCLUSIONS

1. Maintenance of favorable salt balance is essential for planned operation of ground water reservoirs and is of sufficient economic importance to more than justify the necessary expenditures for collection and evaluation of data.

2. The physical and economic problems involved in removal of sufficient water to maintain salt balance from some ground water reservoirs may preclude planned operation of them.

3. Insufficient data are now being collected to evaluate the discernible trends in water quality in many ground water reservoirs and far more data will be required to predict and evaluate salt-balance requirements for the reservoirs under planned operation.

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TABLE I

ALLOWABLE SALT CONCENTRATIONS IN DRINKING WATER*

Constituent	Concentration, in ppm	Constituent	Concentration, in ppm
Mandatory		Permissive	
Lead (Pb)	0.1	Copper (Cu)	3.0
Fluoride (F)	1.5	Iron (Fe)) Manganese (Mn)) together	0.3
Arsenic (As)	0.05	Magnesium (Mg)	125
Selenium (Se)	0.05	Zinc (Zn)	15
Chromium hexavalent	0.05	Chloride (Cl)	250
		Sulphate (SO_4)	250
		Phenolic compounds	0.001
		Total solids - preferred (limit allowed, 1000 ppm)	500

* United States Public Health Service

TABLE II

QUALITATIVE CLASSIFICATION OF IRRIGATION WATER

Factor	Class 1 Excellent to good	Class 2 Good to injurious	Class 3 Injurious to unsatisfactory
Conductance (K x 10^6 at 25°C)	Less than 1000	1000-3000	More than 3000
Boron, ppm	Less than 0.5	0.5-2.0	More than 2.0
Per cent sodium	Less than 60	60-75	More than 75
Chloride, ppm	Less than 178	178-355	More than 355

TABLE III

AVERAGE ANNUAL SUPPLY AND DISPOSAL OF WATER AND QUANTITY OF SALTS ENTERING RESERVOIR

Average annual supply, in acre-feet		Concentration of total salts, in ppm
Surface inflow	80,000	400
Precipitation	100,000	negligible
Subsurface inflow	10,000	500
Subtotal	190,000	

Average annual surface disposal of runoff and precipitation, in acre-feet	
Consumptive use of precipitation	75,000
Consumptive use of applied surface water	40,000
Subtotal	115,000

Net recharge of the reservoir = 190,000 - 115,000 = 75,000 acre-feet

Average annual quantity of salts entering reservoir, in tons		
Surface inflow -	80,000 x 400 x .00136 =	43,500
Subsurface inflow -	10,000 x 500 x .00136 =	6,800
TOTAL		50,300

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A GRAPHICAL SOLUTION FOR FLOW IN EARTH CHANNELS

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(Proc. Paper 1360)

SYNOPSIS

This paper presents a graphical solution utilizing Manning's Formula for the design of earth canals with trapezoidal cross-sections. Work in a designing office charged with the design of numerous sections of earth canals will be facilitated by the use of these graphs. These graphs were prepared for a specific value of the coefficient of channel roughness closely approximating that for earth canals. Graphs for other values of "n" can readily be constructed.

INTRODUCTION

The task of designing trapezoidal earth canals can greatly be simplified and the amount of work reduced through the use of graphs. Among the commonly used formulas readily available for the design of canals are Chezy-Kutter Formula and Manning's Formula.

In the Bureau of Public Works in the Philippines, earth canals used to be designed by using charts which were constructed based on the Chezy-Kutter's Formula. The graphical solution of Manning's Formula as given in this paper permits a rapid solution of the problem and is to be preferred from the available charts utilizing the Chezy-Kutter's Formula from the point of view of simplicity, ease, and rate of calculation. It is observed that agreement between the Chezy-Kutter and Manning's Formula is fairly close under all ordinary working conditions.

Derived Equations from Manning's Formula and Construction of the Charts

In the Bureau of Public Works in the Philippines, most trapezoidal earth

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1. Designing Engr.; Bureau of Public Works, Manila, Philippines.

canals are designed with the following properties:

$n = 0.025$
$b/d = 2.5$
$ss = 1.5:1$

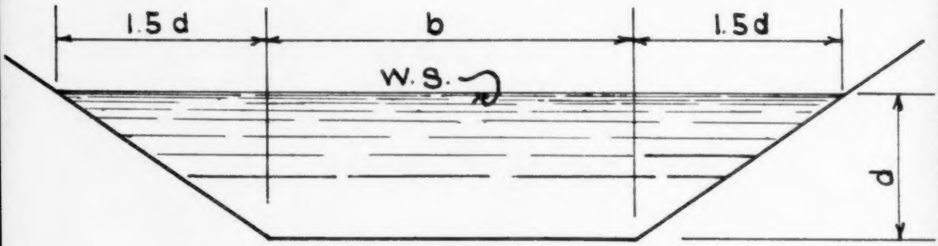


Figure 1 - CANAL SECTION

Manning's Formula is given by the expression

$$Q = \frac{A}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \tag{1}$$

where n = roughness coefficient
A = area of cross-section
Q = discharge
R = hydraulic radius
S = slope of the canal

Upon subtitution of the above-mentioned properties of the canal in equation (1), Manning's Formula is reduced to

$$\frac{Q}{d^{\frac{8}{3}}} = 121 S^{\frac{1}{2}} \tag{2}$$

For a given slope S, equation (2) is further simplified to

$$Q = K d^{\frac{8}{3}} \tag{3}$$

where $K = 121S^{1/2}$
The family of curves representing equation (3) in Graph I (figure 2a) were plotted for various slopes with the discharge Q in cubic meters per second as abscissa and the depth d in meters as the ordinate. One can readily see that the depth d of the canal is easily obtained if the Q and S values are known. For the same properties in Graph I, the equation for the velocity is

$$v = 30.3\, S^{\frac{1}{2}} d^{\frac{2}{3}} \tag{4}$$

The velocity isopleths in meters per second were plotted from equation (4).

From figure 2a Graph I, the depth d of the canal and the accompanying velocity v are obtained if the discharge Q and the slope S are known.

In some special cases however, it may be desirable to use a base-depth ratio other than 2.5. Values other than 2.5 are generally used for canals passing through side hills in order to minimize the excavations. Another factor governing the choice of the base-depth ratio is the slope of the terrain along which the canal is to be laid out. If it is relatively steep, then it will be desirable to reduce the speed of flow. Since the speed is a function of both the slope and the hydraulic radius, if the latter can be reduced then the velocity will decrease proportionately. By adopting a bigger value of the base-depth relationship, the wetted perimeter will decrease which in turn will decrease the hydraulic radius. For relatively flat slopes, b/d values slightly less than 2.5 is used in order to increase the hydraulic radius. By such a procedure the velocity is increased while keeping the slope unchanged.

Graph II (figure 2b) was designed for variable base-depth relationship. Values of b/d from 1 to 3.5 were plotted. For n = 0.025 and ss = 1.5:1, Manning's formula is reduced to equation (5)

$$\frac{Qn}{d^{\frac{8}{3}} S^{\frac{1}{2}}} = \frac{(1.5+k)^{\frac{5}{3}}}{(3.6+k)^{\frac{2}{3}}} \tag{5}$$

where k = base-depth ratio.

The working equation for Graph II then becomes

$$\left(\frac{1}{d^{\frac{8}{3}}}\right)\left(\frac{Q}{S^{\frac{1}{2}}}\right) = \frac{(1.5+k)^{\frac{5}{3}}}{(3.6+k)^{\frac{2}{3}} n} \tag{6}$$

This equation was worked out for various values of "k" and the family of curves plotted in Graph II. The ordinate "d" is in meters and the abscissa is $(Q/S^{\frac{1}{2}})$. Values of b/d other than those drawn in Graph II can be obtained by interpolation between the two successive curves.

Illustrative Examples

The use of the 2 graphs will be shown by the following two examples.

EXAMPLE 1

This problem will illustrate the use of Graph I.

Given: Q = 3.5 cubic meters per second
b/d = 2.5
S = 0.0005

Required:

the depth "d" and the velocity "v"

Solution: Follow through a vertical line from the given value of Q equals 3.5 till it intersects the curve for $S = 0.0005$. Read the ordinate corresponding to the point of intersection and the velocity by interpolating between the two velocity lines. In Graph I, the depth "d" equals 1.10 meters and the velocity "v" equals 0.72 meter per second.

EXAMPLE 2

This will illustrate the use of Graph II.

Given: $Q = 3$ cubic meters per second
$b/d = 3$
$n = 0.025$
$ss = 1.5:1$
$S = 0.0003$

Required: depth "d"

Solution: $Q/S^{1/2} = 3/0.0173 = 173.5$

Follow through a vertical line passing thru the value of 173.5 in the abscissa till it intersects the curve marked $b/d = 3$. The ordinate of the point will be the value of the required depth "d." In this problem d equals 1.08 meters. Since the area A equals $4.5d^2$ for $b/d = 3$, the velocity will be 0.57 meter per second.

CONCLUSION

The solution of the 2 illustrative examples will show the ease of application of the 2 graphs in relation to the design of trapezoidal earth canals. For canals of different cross-sections and for values of the roughness coefficient other than the value used in this paper, similar graphs can readily be constructed.

APPENDIX

Tabulated values are given in this appendix in order that anyone who may desire to use this graphical solution in canal designs can readily construct these graphs for office use. The tabulated values for figure 2a are given in Tables I and II, while that for figure 2b are in Table III.

TABLE I. DISCHARGE IN CUBIC METERS PER SECOND

d \ b/d	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50
0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3
0.2	0.9	1.0	1.2	1.3	1.4	1.6	1.7	1.8	1.9	2.1	2.2
0.3	2.7	3.0	3.4	3.7	4.1	4.5	4.8	5.2	5.7	5.9	6.3
0.4	5.8	6.6	7.3	8.1	8.9	9.7	10.5	11.3	12.1	12.9	13.8
0.5	10.4	11.8	13.2	14.7	16.1	17.5	19.0	20.4	21.9	23.4	24.9
0.6	17.0	19.3	21.6	23.9	26.2	28.5	31.0	33.3	35.7	38.1	40.5
0.7	25.6	29.0	32.5	36.1	39.5	43.0	46.6	50.2	53.9	57.5	61.1
0.8	36.7	41.5	46.5	51.6	56.5	61.5	66.8	71.8	77.0	82.2	87.5
0.85	43.0	48.7	54.5	60.5	66.4	72.2	78.4	84.2	90.5	96.5	102.5
0.9	50.2	56.8	63.5	70.5	77.4	84.2	91.5	98.0	105.0	112.5	120.0
0.95	57.9	65.5	73.2	81.0	89.3	97.0	105.5	113.0	121.5	130.0	138.0
1.0	66.5	75.2	84.2	93.4	102.4	111.5	121.0	130.0	139.5	149.0	158.0
1.05	75.7	85.6	95.9	106.0	117.0	127.0	138.0	148.0	159.0	170.0	180.0
1.1	85.7	97.0	108.5	120.5	132.0	144.0	156.0	168.0	180.0	192.0	204.0
1.15	96.4	109.0	122.0	135.5	148.5	161.5	175.5	188.5	202.0	216.0	230.0
1.2	108	122	137	152	167	182	197	212	228	243	258
1.25	120	136	152	169	185	202	219	235	252	270	287
1.3	133	151	169	188	206	224	243	262	280	300	318
1.35	148	168	188	208	228	248	270	290	311	332	353
1.4	163	184	206	229	251	273	296	318	342	365	388
1.45	179	202	226	252	276	300	326	350	375	400	426
1.5	196	222	248	276	302	329	357	384	412	440	467
1.55	214	242	271	301	330	359	390	418	450	480	510
1.6	233	263	294	327	359	390	423	455	488	522	555
1.65	253	286	320	355	389	424	460	494	530	566	602
1.7	274	310	347	385	422	460	499	535	575	614	654
1.75	296	335	375	416	456	496	539	579	620	662	705
1.8	318	360	403	447	491	534	580	622	668	714	759
1.85	343	388	435	483	529	575	625	670	720	769	817
1.9	386	416	466	517	568	617	670	720	772	825	878
1.95	394	446	499	554	607	660	717	770	826	883	940
2.0	422	478	535	594	650	708	769	825	885	945	1005
2.05	451	510	570	634	694	755	820	880			
2.1	481	543	608	675	740	806	874				
2.15	513	579	648	720	789	859	930				

TABLE II. DISCHARGE IN CUBIC METERS PER SECOND

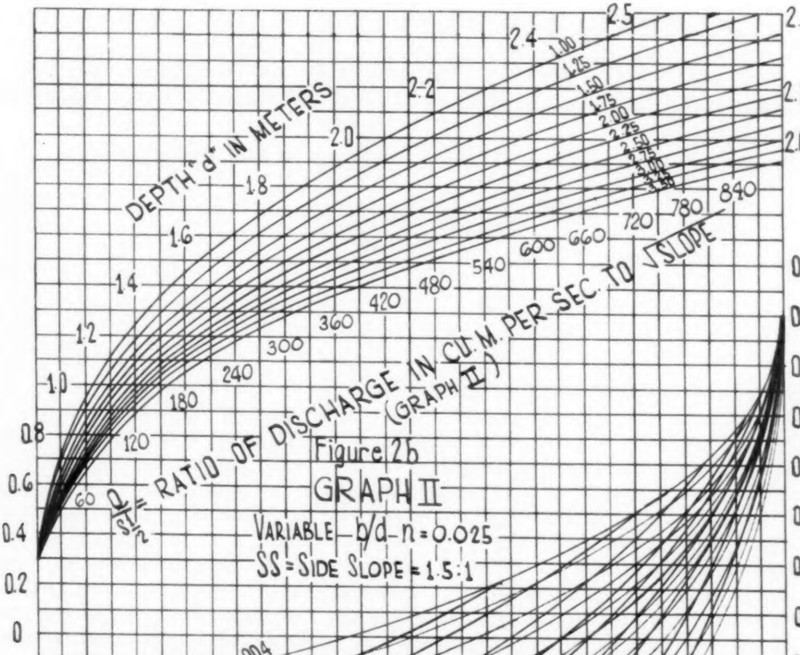
d \ S	0.0001	0.000125	0.0002	0.00025	0.0003	0.00035	0.00040	0.00045	0.00050	0.00060
0.1	0.002	0.003	0.003	0.004	0.004	0.005	0.005	0.005	0.005	0.006
0.2	0.017	0.018	0.024	0.027	0.029	0.032	0.034	0.036	0.038	0.042
0.3	0.048	0.054	0.068	0.076	0.083	0.091	0.097	0.105	0.108	0.118
0.4	0.105	0.117	0.149	0.166	0.181	0.200	0.210	0.220	0.240	0.260
0.5	0.19	0.21	0.27	0.30	0.33	0.36	0.38	0.40	0.42	0.46
0.6	0.31	0.34	0.44	0.49	0.53	0.58	0.62	0.66	0.69	0.76
0.7	0.46	0.52	0.66	0.74	0.80	0.87	0.93	0.99	1.04	1.14
0.8	0.67	0.74	0.94	1.05	1.15	1.25	1.34	1.41	1.49	1.64
0.85	0.78	0.87	1.11	1.24	1.35	1.47	1.57	1.66	1.75	1.92
0.9	0.91	1.01	1.29	1.44	1.57	1.71	1.83	1.93	2.04	2.23
0.95	1.05	1.17	1.49	1.66	1.81	1.97	2.11	2.23	2.35	2.58
1.00	1.21	1.34	1.71	1.91	2.08	2.26	2.42	2.56	2.70	2.96
1.05	1.38	1.53	1.95	2.18	2.37	2.58	2.76	2.92	3.08	3.38
1.10	1.56	1.73	2.21	2.46	2.68	2.92	3.12	3.30	3.48	3.82
1.15	1.75	1.94	2.48	2.77	3.02	3.28	3.51	3.71	3.92	4.29
1.20	1.97	2.18	2.78	3.11	3.39	3.68	3.94	4.17	4.41	4.83
1.25	2.19	2.42	3.09	3.46	3.76	4.09	4.38	4.63	4.89	
1.30	2.43	2.69	3.43	3.84	4.18	4.55	4.86	5.14		
1.35	2.70	2.99	3.81	4.26	4.64	5.04	5.40			
1.40	2.96	3.28	4.19	4.68	5.10	5.54				
1.45	3.26	3.60	4.60	5.14	5.60					
1.50	3.57	3.95	5.04	5.64						
1.55	3.90	4.32	5.50							
1.60	4.23	4.69								
1.65	4.60	5.09								
1.70	4.99									

CONTINUATION OF TABLE II.

d \ S	0.00070	0.00080	0.00090	0.0010	0.00125	0.0015	0.00175	0.0020	0.0030	0.0040
0.1	0.006	0.007	0.007	0.008	0.008	0.009	0.010	0.011	0.013	0.015
0.15	0.019	0.020	0.022	0.023	0.026	0.028	0.030	0.032	0.040	0.046
0.2	0.045	0.048	0.051	0.054	0.060	0.065	0.071	0.076	0.092	0.107
0.25	0.079	0.085	0.091	0.095	0.106	0.117	0.126	0.135	0.161	0.191
0.3	0.128	0.14	0.15	0.15	0.17	0.19	0.20	0.22	0.26	0.31
0.35	0.19	0.21	0.22	0.23	0.26	0.28	0.31	0.33	0.40	0.47
0.4	0.28	0.30	0.32	0.33	0.37	0.41	0.44	0.47	0.57	0.66
0.45	0.38	0.41	0.43	0.46	0.51	0.56	0.60	0.64	0,78	0.91
0.5	0.50	0.54	0.57	0.60	0.67	0.74	0.80	0.85	1.05	1.20
0.55	0.65	0.70	0.74	0.77	0.86	0.95	1.02	1.09	1.34	1.55
0.6	0.82	0.88	0.93	0.98	1.09	1.20	1.29	1.38	1.69	1.96
0.65	1.01	1.10	1.15	1.21	1.35	1.48	1.60	1.71	2.09	2.42
0.7	1.23	1.32	1.40	1.48	1.64	1.81	1.95	2.08	2.55	2.94
0.75	1.48	1.59	1.68	1.77	1.98	2.17	2.34	2.50	3.06	3.54
0.8	1.76	1.89	2.00	2.11	2.35	2.58	2.79	2.98	3.64	4.22
0.85	2.07	2.22	2.35	2.47	2.76	3.03	3.27	3.50	4.28	4.94
0.9	2.41	2.58	2.74	2.88	3.21	3.53	3.81	4.07	4.98	
0.95	2.78	2.98	3.16	3.32	3.71	4.08	4.40	4.70		
1.00	3.19	3.42	3.63	3.82	4.26	4.68	5.05			
1.05	3.64	3.90	4.14	4.36	4.85					
1.10	4.12	4.41	4.68							
1.15	4.63	4.96								

TABLE III.. DEPTH IN METERS

v \ S	0.0001	0.00015	0.0002	0.0003	0.0004	0.0005	0.0007	0.0009	0.00125	0.0015	0.002	0.003
0.10	0.19	0.14	0.11									
0.20	0.54	0.40	0.32	0.23	0.19	0.16						
0.25	0.75	0.55	0.44	0.33	0.26	0.22						
0.30	0.98	0.73	0.58	0.43	0.35	0.30	0.23	0.19	0.15			
0.35	1.25	0.92	0.74	0.54	0.44	0.37	0.29	0.24	0.19	0.16	0.13	0.10
0.40	1.52	1.12	0.90	0.66	0.54	0.46	0.35	0.29	0.23	0.20	0.16	0.12
0.45	1.82	1.33	1.07	0.79	0.64	0.54	0.42	0.35	0.27	0.24	0.19	0.14
0.50	2.12	1.57	1.25	0.92	0.75	0.64	0.49	0.41	0.32	0.27	0.22	0.17
0.55		1.82	1.45	1.06	0.86	0.73	0.57	0.47	0.37	0.32	0.26	0.19
0.60			1.66	1.22	0.99	0.84	0.65	0.54	0.41	0.36	0.29	0.22
0.70				1.53	1.25	1.05	0.82	0.68	0.53	0.46	0.37	0.27
0.80					1.52	1.29	1.00	0.83	0.65	0.56	0.45	0.33
0.90						1.53	1.18	0.99	0.77	0.67	0.54	0.40
1.00								1.15	0.90	0.78	0.63	0.47
1.10								1.33	1.04	0.91	0.73	0.54
1.20									1.18	1.03	0.83	0.61
1.30									1.35	1.17	0.94	0.69
1.40											1.04	0.77


DEPTH "d" IN METERS
$\frac{Q}{S^{1/2}}$ = RATIO OF DISCHARGE IN CU. M. PER SEC. TO √SLOPE (GRAPH II)
Figure 2b
GRAPH II
VARIABLE b/d n = 0.025
SS = SIDE SLOPE = 1.5:1
DEPTH "d" IN METERS
Q = DISCHARGE IN CU. M. PER SEC. (GRAPH I)
Figure 2a
GRAPH I
n = 0.025
SS = 1.5:1
b/d = 2.5

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COMMON ERRORS IN MEASUREMENT OF IRRIGATION WATER

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(Proc. Paper 1362)

SUMMARY

Devices and structures in general use in the United States for measuring irrigation water are usually subjected to changes in water levels upstream, and perhaps downstream, from the point of measurement. The generally accepted approach to meet this problem is standardization and calibration of the measuring equipment. Use of tables, graphs, or charts developed from the calibration for determining discharge in the field is based on the criteria that the field structure is a replica of the device from which the data were derived and that the flow conditions are identical. Deviations from these standards will result in errors. The magnitude of errors resulting from changes in certain dimensions, incorrect settings, changes in flow patterns, and other deviations is evaluated for some of the commonly used measuring devices. It is concluded that measurements obtained from equipment which is capable of operating with a high degree of accuracy may be subjected to gross errors unless due care is exercised in fabrication, installation, operation, and maintenance.

INTRODUCTION

Many of the measurements of flow made in irrigation systems may not be as accurate as assumed or as would be expected from the type of device or structure being employed. In all probability the order of accuracy of the flow measurements derived from the many types of devices used in irrigation systems will show some variation. This deviation will usually be more evident between the different types of structures and methods used but may be apparent when units of the same type are compared.

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Some of the reasons for the deviations in accuracy are: (1) principle of operation (that is, volumetric, velocity, or other); (2) degree of exactness to which the flow coefficients have been established (that is, how much study and analysis have been made on the particular device or method); (3) workmanship and the care that is exercised in construction or fabrication and in installation (including the setting); (4) adaptability of the particular device to the existing conditions; (5) proper setting with respect to flow conditions (that is, the approach and exit flow conditions); (6) proper maintenance; (7) cost (in general, better accuracy may be obtained from the more costly devices, although this is not always true); (8) manner of obtaining the final result (that is, whether the rate of flow is determined directly from the device or method, or whether, for instance, head is measured and the end result obtained by calculation or from tables, charts, or curves); (9) means used for obtaining the needed intelligence, for instance, use of a staff gage in the flow, a gage in a stilling well, or a continuous recorder for obtaining head; (10) care exercised in obtaining the intelligence; and (11) other factors such as extraneous material in the water, range of flows to be covered, etc.

It is not probable that absolute accuracy can be obtained in all instances. However, the reduction of errors to a minimum may be possible if all factors are taken into consideration.

Generally speaking, errors are of two classes: (1) avoidable errors which result from carelessness and can be eliminated by thorough supervision and strict attention to details, and (2) unavoidable errors which are errors of degree; and although possibly they cannot be completely eliminated, they can, by exercise of extreme care and knowledge of their nature and magnitude, be alleviated and satisfactory overall results obtained.

In the United States numerous devices and structures have been developed for the measurement of flows in irrigation systems. Nearly all of these developments are designed to operate in conjunction with separate equipment to control the flows. Only a few of them serve the dual purpose of control and measurement. Except in rare cases, the control is manually operated.

The design of essentially all of the open channel irrigation systems is such that fluctuations of the water levels both upstream and downstream of the measuring device are tolerated. The measuring devices have been developed to accommodate this design procedure. In effect, any change in water level upstream and, generally, downstream is reflected as a change in discharge through the measuring device.

The generally accepted approach to the solution of the problem of fluctuating water levels is the standardization and calibration of the measuring equipment. The result is a device, which, when built and installed in accordance with the established standards, will pass a range of known discharges for a range of upstream and downstream water levels. The exact instantaneous discharge can be determined by observing the upstream and downstream heads, by means of suitable gages, correctly referenced, and entering charts, graphs, or tables which have been developed by prior calibration of the device under carefully controlled conditions.

Such a procedure assumes that the field installation is a suitable replica of the installation which was calibrated, usually in a hydraulic laboratory. Further assumptions are that conditions of flow, especially in regard to velocity distribution patterns, are similar and that the heads, and other necessary measurements, can be determined with a comparable degree of accuracy.

Since the discharge tables, charts, or graphs are the results of calibrations, they are based on empirical relationships and not on a rational analysis in all instances. Therefore, they are not necessarily susceptible to accurate extrapolation beyond the range of observations from which they were developed.

To obtain accurate measurements of flows in an irrigation system by means of those devices now in general use in the United States, it is therefore necessary to know something of the standards developed and conditions of calibration.

The intent of this discussion is to point out some of the possible errors in measurement which will result from disregard of adherence to close tolerances in the standards developed; from failure to make accurate observations; and from other deficiencies. The examples cited are devices used in open channel irrigation systems. Similar arguments are generally applicable to equipment and structures used in closed conduit systems.

The magnitude of errors introduced in flow measurements by departure from established criteria can be evaluated in many cases. In other instances, errors or inconsistencies are evident but exact evaluation is difficult.

The devices mentioned are not necessarily those most susceptible to errors nor are the cases cited all of those which may cause errors in measurements. Weirs are widely used in the United States for the measurement of irrigation and drainage water, therefore, more attention has been given to possible errors in this device. Since broad crested weirs are not generally used in irrigation measurements, the comments are directed toward sharp crested weirs.

The author is mindful of the general practice on irrigation systems of reading the head gages on measuring devices at intervals, usually once a day. Since an irrigation system rarely reaches regimen flow conditions because of fluctuations in the source of supply, changes in demands, etc., this practice may result in major errors. However, this is a moot question among operators. Until such times as a continuous recorder is developed which will be economically feasible for installation this practice will be followed because it is not practicable to obtain a large number of readings each day at each of the points of measurement. However, the errors pointed out in the following discussion are of such magnitude that they merit careful consideration.

Sources of Error

Faulty Fabrication or Construction

There are numerous possibilities for introduction of errors in flow measurements resulting from faulty construction of measuring devices. The error caused by incorrect dimensions of some of the structures is readily evaluated and may be used as an example to demonstrate the error.

Table 1 shows the discharge error for rectangular or Cippoletti weirs for an incorrect measurement of length of weir crest of only 0.01 foot as compared to the standard which was calibrated and from which the flow formula was derived. Since discharge is directly related to length in the flow equation, an error in length of 0.05 foot would cause the discharge to be in error by five times the values shown in the table for any observed head.

The discharge error caused by 0.01-foot error in measurement of the width of the throat of Parshall flumes of standard widths from 1 to 4 feet is

TABLE I
DISCHARGE ERROR FOR RECTANGULAR OR CIPPOLETTI WEIR FOR AN INCORRECT MEASUREMENT OF LENGTH

$$Q = CLH^{3/2}$$

$$Q' = C(L+\Delta L)H^{3/2}$$

$$\frac{Q'}{Q} = \frac{L+\Delta L}{L}$$

$$Q' = \left(\frac{L+\Delta L}{L}\right)Q$$

Error of 0.01 foot in length measurement

WEIR LENGTH L FEET	% ERROR $\frac{Q'-Q \times 100}{Q}$
1.0	1.0
2.0	0.5
3.0	0.33
4.0	0.25

shown in Table 2. A constant head of 0.2 foot has been assumed. Also shown in this table is the error introduced by faulty measurement of 0.02 and 0.03 foot. The error is essentially constant for different values of measured head. The flow equation used in development of the table was derived empirically from calibrations.(1)

Similarly, it may be shown that an error in measurement of the width or the breadth of a rectangular submerged orifice will cause considerable error in discharge. Since the discharge is directly related to the area of the orifice, the magnitude of the error is similar to that for the weir, and an error in either length or breadth measurement will be constant for various heads on the orifice or gate.

Error in Discharge Measurements Due to Transverse Slope of Weir Crest

When installing Cippoletti and rectangular weirs in the field, it is necessary to set the crest exactly horizontal. If it is known that the crest is not level it is common practice to consider the effective head to be the average head on the weir. The error caused by this practice is shown in Figure 1.

Actually, a more precise method is to calculate the discharge using the head at the low end and the head at the high end and average the discharge derived from these two calculations.(2) If this is done, the error is reduced to minor significance.

If it is not known that the weir is inclined and the gage zero is referenced to either the high or the low end, the resulting error is considerably greater. Figure 2 shows the magnitude of the error in discharge resulting from measurement of head at either end of 12-, 24-, 36-, and 48-inch Cippoletti weirs having a transverse slope of 0.01 instead of using the average head over the weir. An inclination of about 6° will cause an error in the order of 1 percent. An angle of this magnitude should be detected by eye and corrective measures taken.

Error in Discharge Resulting from Errors in Reading the Head

Perhaps the most common error in measuring irrigation water is to misread the head. This may result from incorrect location of the gage, or because the head gage is dirty, a stilling well is not used, and there is considerable fluctuation of the water surface, or carelessness on the part of the reader in not obtaining a good average reading at the time the gage is observed.

Figure 3 shows the error in discharge resulting from a 0.01 foot incorrect head reading on 12- to 48-inch Cippoletti and 90° V-notch weirs. This figure clearly illustrates that even with a small error of 0.01 foot, an error of approximately 7-1/2 percent in discharge results when the lower heads are being measured. For greater heads, the error is less. Also, it can be noted that for the longer weirs this slight error in head reading results in quite large errors in discharge measurements.

As in the case of weirs, the head at the throat of a Parshall flume is quite easily misread in the field. Figure 4 shows the error in discharge measurements resulting from a misreading of the gage of only 0.01 of a foot. Parshall flumes of throat widths of from 6 to 36 inches are shown on this figure. It

TABLE 2
DISCHARGE ERROR FOR INCORRECT MEASUREMENT OF WIDTH OF PARSHALL FLUME

Equation $Q = 4WH_a^{1.522W^{0.026}}$

W ft.	H_a ft.	Q cfs	Q′ cfs	% ERROR $\frac{Q'-Q}{Q} \times 100$
Error in width measurement of 0.01 foot				
1.0	0.2	0.348	0.351	0.86
2.0	0.2	0.656	0.659	0.45
3.0	0.2	0.972	0.975	0.30
4.0	0.2	1.264	1.267	0.23
Error in width measurement of 0.02 foot				
1.0	0.2	0.348	0.355	2.0
Error in width measurement of 0.03 foot				
1.0	0.2	0.348	0.359	3.1

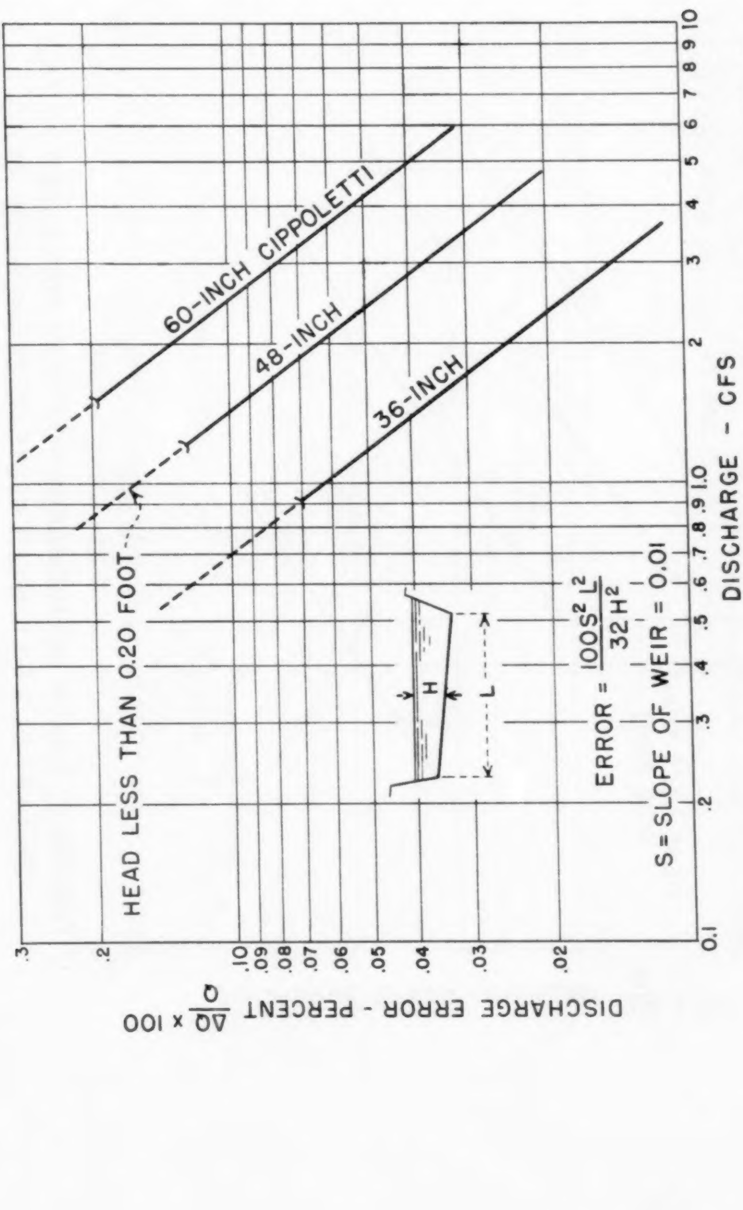


FIGURE 1 - ERROR IN DISCHARGE DUE TO TRANSVERSE SLOPE OF CIPPOLETTI WEIR CREST

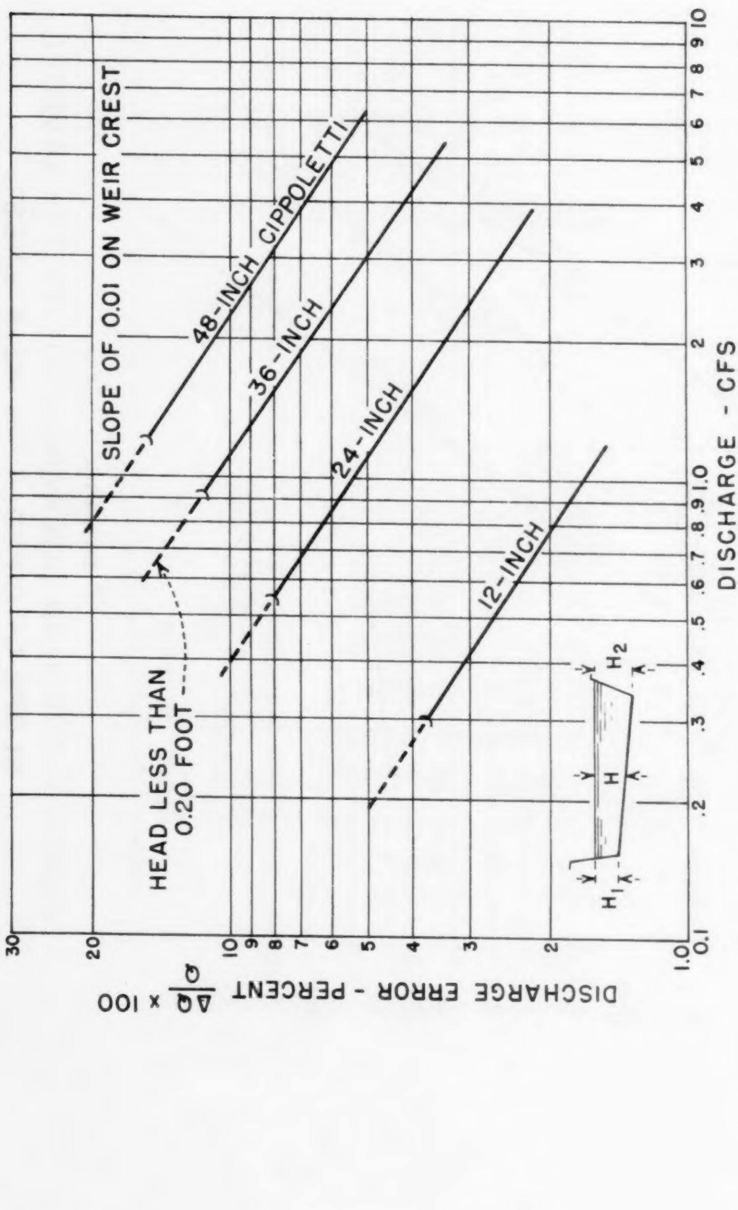


FIGURE 2-ERROR IN DISCHARGE FOR HEAD MEASURED AT END OF CIPPOLETTI WEIR INSTEAD OF AVERAGE HEAD

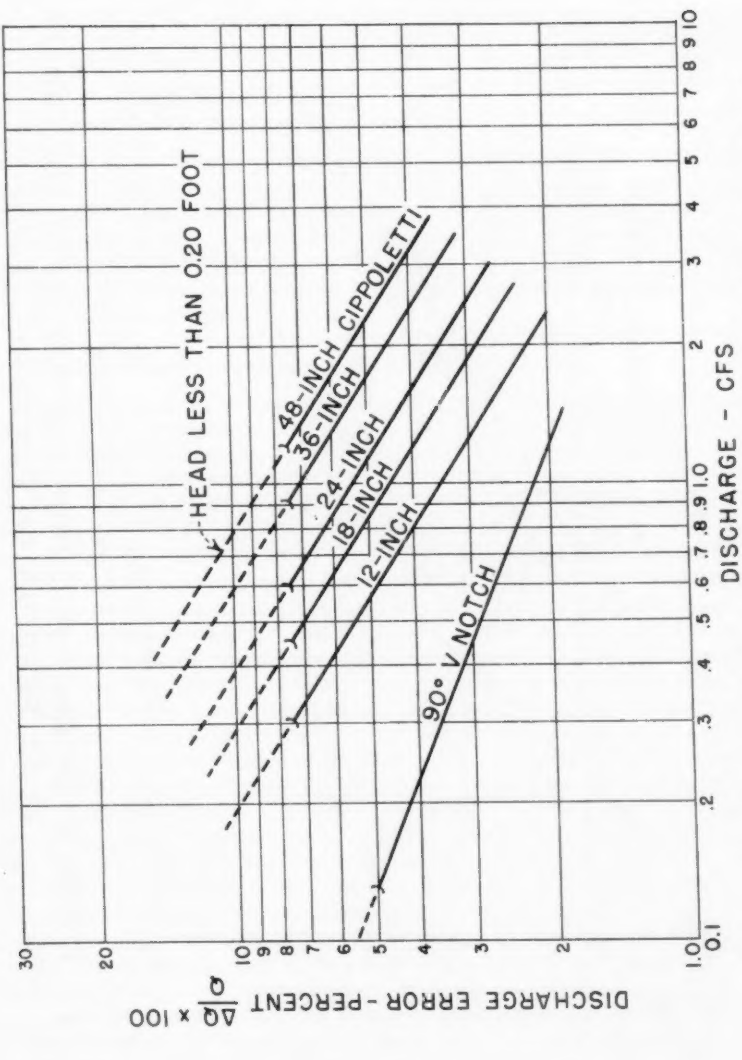


FIGURE 3-DISCHARGE ERROR FOR 0.01-FOOT INCORRECT HEAD READING ON WEIRS

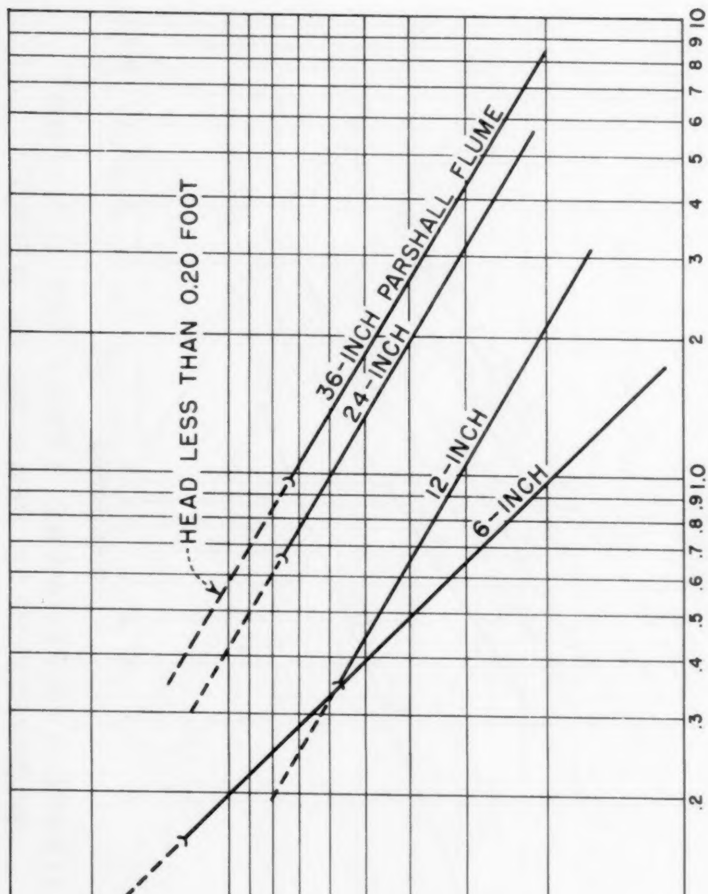


FIGURE 4 - DISCHARGE ERROR FOR 0.01-FOOT INCORRECT HEAD READING ON PARSHALL FLUMES

can be noted that the error, in general, is approximately the same as for misreading the head an equal amount when weirs are considered.

Figure 5 shows the error in measurement resulting from an error of 0.01 foot in reading of the head on an 8-, 12-, and 18-inch meter gate and an 18-inch screw-lift gate. The meter gate has a circular leaf and the screw-lift gate has a rectangular leaf. Included on Figure 5 is the error caused by 0.01 foot incorrect head reading for the constant-head orifice turnout.[3] It should be noted that the percentage error in discharge resulting from misreading the head on an orifice is in general less than the same misreading on a weir.

Error in Discharge Measurement
Caused by Incorrect Zero Setting

The error for incorrect zero setting of the head gage is of the same magnitude as the error for misreading the head an equal amount. Improper positioning of the gage used to read the head is probably the most common error found. In the field it is difficult to reference the exact zero of the gage to the crest of a weir, a submerged orifice, or to a turnout gate. Extreme care should be exercised in setting the gages since incorrect settings cause errors at all flows, Figures 3, 4, and 5.

Errors Resulting From Improper Gage Location

Proper location of the gage for obtaining head on measuring devices is important if errors are to be avoided. In most instances flow relationships have been determined empirically with a particular type of gaging device placed in a specific location. Hence, there is included in the overall calibration a secondary effect of calibration of the gaging system employed to obtain head. Because of changes in the flow pattern of the stream as it passes through the measuring section, minor deviations from the standard in gage design and location may appreciably affect the quality of the measurements.

In the case of a weir, there is a downward curve of the water surface as the flow passes through the notch. This curved surface, or drawdown, extends some distance upstream. The exact distance is dependent upon local conditions. The head of the weir must be measured beyond the effect of the drawdown. In the development of the basic weir formulas, the head was observed at distances upstream from the weir notch varying from about 4 to 9 times the maximum head over the weir. Therefore, many authorities have accepted a minimum distance of four times the maximum head to be measured. However, King[4] says the distance should be at least 2.5 times the maximum head. Experiments have shown that there is some effect of drawdown to a distance upstream of some six times the head on the weir.[5,6] However, the influence at this distance is minor. Within the practical limits of the gages used at weir installations in irrigation systems, it appears that a distance upstream of four times the maximum head is quite adequate providing other criteria such as height of weir, width of weir pool, etc., are complied with.

Unpublished results of brief studies conducted in the Hydraulic Laboratory of the Bureau of Reclamation show that it is extremely difficult to detect differences of head on enameled staff gages located 2, 4, and 10 times the head on the weir. These same studies did show that positioning the enameled staff gage on the weir bulkhead, a practice sometimes followed in irrigation measurements, Figure 6, may result in errors. Certain positions on the bulkhead,

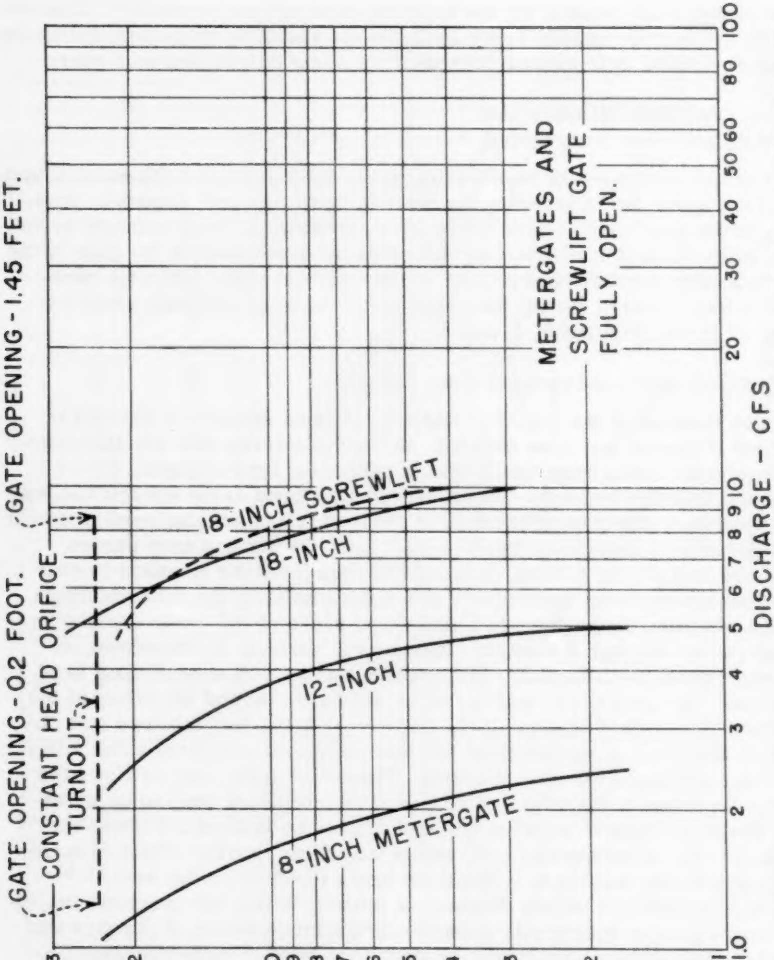


FIGURE 5-DISCHARGE ERROR FOR 0.01-FOOT INCORRECT HEAD READING ON GATES

with respect to the weir notch, gave a higher reading for certain flows than a gage correctly positioned. At other flows the reading was less. As the gage on the bulkhead was moved away from the weir notch more consistent results were evident. It was found, for the flow conditions tested, that when the gage was placed on the weir bulkhead at a minimum distance of twice the maximum recommended head for the weir, the difference in the heads read on this gage and one correctly placed upstream was within the limits of visual observation.

When the velocity of approach is high and the irrigation channel has a high loss coefficient, there is a danger of placing the gage so far upstream from the weir crest that an error will prevail unless a correction is made for the loss of head due to channel friction between the point of measurement and the weir.

Errors in measurement can easily occur if the gages used in a Parshall flume are not placed in the manner and location developed in the standards. The ratings for this flume include a calibration of the gage positions. The gages are located in drawdown areas. Under these conditions, movement of the gage upstream or downstream from the standardized location will change the head reading and an error in discharge will result. For similar reasons, if a stilling well is used, the type and location of the entrance to the wells should be as specified. Substantial errors in field measurements have been traced to changes in location or design of the still well entrances.

Similar remarks apply to the location of the two gages used in the constant-head orifice turnout. The discharge tables developed from the calibration of this device are accurate only if the gages are placed in the locations given in the standard drawings.

Discharge Errors Due to Neglecting Velocity of Approach to a Weir

In practical application the cross sectional area of the approach channel can usually be made sufficiently large in comparison with the weir notch to render the effect of velocity of approach negligible. If, however, the approach velocity is not maintained at or below 0.5 foot per second, it must be taken into account and a correction applied.(3) In other words, if the normally used equations, charts, or tables are used, without correction, for obtaining discharge from measured head an error will result.

In irrigation practice the velocity of approach to a weir is usually increased over that for which it was originally designed by: (1) a general restriction of the cross sectional area of the weir pool by deposits of vegetal growth, or (2) sediment or other accumulations in the bottom of the weir pool. Either will change the standards to which the weir installation should conform.

A general reduction of the cross sectional area of the weir pool will cause an increase in approach velocity which is directly related to the degree of restriction. The percentage error for a range of approach velocities and heads over weirs, except the V-notch type, is given in Table 3. The error is such that the discharge is actually greater than that obtained from the discharge tables by the percentages given in Table 3.

Authorities agree that the crest of the weir should be a distance not less than two times the depth of water over the crest above the bottom of the approach channel for accurate results. A greater height of weir crest is to be preferred when practicable. A weir installed in an irrigation channel in

Figure 6. Enameled gage for observing head on weir fastened to weir bulkhead too close to weir notch.

TABLE 3
DISCHARGE ERROR RESULTING FROM FAILURE TO CORRECT FOR VELOCITY OF APPROACH

Velocity of Approach ft./sec.	Observed Head Over Weir-Feet				
	0.2	0.4	0.6	0.8	1.0
	(Error in per cent)				
0.5	2.7	1.3	0.9	0.6	0.6
1.0	9.8	5.1	3.4	2.7	2.2
1.5	20.8	10.9	7.5	5.7	4.7
2.0	33.5	18.1	12.6	9.7	7.9
2.5	48.0	26.6	18.7	14.5	11.9
3.0	63.7	36.1	25.6	19.9	16.5

accordance with this standard may retain its accuracy for only a short period because of reduction of depth of the weir pool by sediment deposits, Figure 6. The regularly used tables will no longer apply. The error may be reduced or possibly eliminated by use of Rehbock's formula for computing discharge from the head observations.

Table 4 gives the percentage error in discharge that will occur if regular weir tables are used instead of correcting for the reduced height of weir by use of the Rehbock formula. The table is divided into two parts: The first part shows a constant head of 0.2 foot over a weir. The value of the ratio of H over P is varied and the error shown. The second part of the table is calculated for a constant head of 0.5 foot and is handled in a manner similar to the first part of the table.

This error is introduced in the field by improper maintenance and cleaning of the weir pool. As the pool fills the ratio of H over P increases and the error increases.

Numerous instances have been noted in the field where weirs have been placed in channels having relatively high gradients. It is very difficult to hold a properly proportioned weir pool under these conditions and obtain smooth flow through the weir notch. Obviously, the increased velocity of approach and turbulence will cause errors in measurement. Channel curvature and consequent pool velocity distribution over the weir crest will also cause excessive errors which are not easily evaluated. Laboratory experiments have shown that the extreme difference in discharge over a weir for a constant head, but with the upstream velocity distribution varied, amounted to 26 percent.(7) A weir with very poor approach conditions is shown in Figure 7.

Discharge Error Due to Turbulence and Surges

Turbulence and surges occur in approach channels to weirs and other types of measuring devices. The cause is usually high velocity of approach but may be from gates or valves, sudden changes in section, or others. Such disturbances are usually evidenced by erratic results in measurements. The disturbances on the surface rarely follow a true sine wave pattern. Hence, an average reading of the head may cause appreciable error. Since the pattern is very complex, corrections are not readily applied to the calculations. Corrective measures to quiet the flow provide the best solution. This may not be an easy task.

Weir Blade Sloping Upstream or Downstream

In constructing a weir, it is necessary to have the plane of the upstream face of the weir vertical if accurate measurements are to be obtained. Experiments with sloping weirs show that the coefficient changes if the weir blade is tilted in an upstream or downstream direction; that is, when the face of the weir blade is not plumb. This change is slight, and the weir face may be out of plumb a few degrees before the accuracy of the measurement is seriously affected.(8)

Roughness of Upstream Face of Weir and Bulkhead

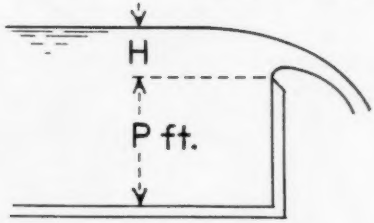
For consistent and accurate flow measurements the upstream face of the bulkhead and weir blade must be smooth. Offsets, protruding bolt heads, and surface roughness must be avoided upon installation. Maintenance is

TABLE 4
ERROR IN DISCHARGE FOR CHANGES IN HEIGHT OF WEIR

Rehbock formula for rectangular sharp crested weir

$$Q = \frac{2}{3}\sqrt{2g}\,LH^{3/2}\left(0.605 + \frac{1}{320H-3} + 0.08\frac{H}{P}\right)$$

$$Q = KLH^{3/2}; \quad K = 3.235 + \frac{1}{60H-0.56} + 0.428\frac{H}{P}$$



WEIR HEIGHT P	$\frac{H}{P}$	COEFFICIENT K	% ERROR $\frac{K-K\infty \times 100}{K\infty}$
Head = 0.2 foot			
0.5	0.4	3.49	5.6
1.0	0.2	3.41	2.7
2.0	0.1	3.37	1.5
3.0	0.07	3.35	0.9
∞	0	3.32	0
Head = 0.5 foot			
0.5	1.0	3.70	13.1
1.0	0.5	3.48	6.4
2.0	0.25	3.38	3.4
3.0	0.17	3.34	2.1
∞	0	3.27	0

Figure 7. Weir with very poor approach conditions.

necessary to retain a smooth surface. Sufficient work has not been done to provide an exact evaluation of the errors resulting from the many possible kinds of roughness. It was found from one series of experiments[7] that "the percentage increase in discharge due to changing the roughness of the upstream face of the weir bulkhead from that of a polished brass plate to that of a coarse file for a distance of 12 inches below the crest is shown to range from about 2 percent for 0.50-foot head to about 1 percent for 1.35-foot head." Nagler's[9] experiments showed that when the upstream face of the weir was roughened, to the crest, with coarse sand (retained on No. 8 standard sieve and passing No. 4) that the increase in discharge ranged from 6.5 percent at a 0.2-foot head to 4.7 percent for a 0.5-foot head. The larger projections caused by the addition of nuts and pieces of metal on the bulkhead below the crest in Nagler's experiment caused about the same increase in discharge.

Rounding of Sharp Edge at Crest of Weir

In irrigation practice many of the older weirs were constructed of wood. In this type of construction the original sharp edge of the crest soon becomes rounded. Ruse and corrosion also produce a rounding effect on metal weir blades. The effect of this rounding is to cause an increase in the flow rate for a given head when compared to a sharp crested weir. Considerable experimentation has been done to evaluate the effect of the rounding of the crest. The results show that the percentage increase in discharge due to the rounding decreases as the head increases. For a head of 0.5 foot, an increase of some 2, 3, 5-1/2, 11, and 13-1/2 percent may be expected for roundings having radii of 1/24, 1/8, 1/4, 1/2, and 3/4 inch, respectively. There is a deficiency of data for the higher heads with the longer radius roundings. However, with radii smaller than those given above, the increases become consistently smaller as the head increases. As an example, the increase in discharge of 2 percent, given above for the 1/24-inch rounding at 0.5-foot head, becomes 0.7 percent at 1.0-foot head and about 0.5 percent at 1.35-foot head. An extreme example of rounding of a weir crest is shown in Figure 8.

Submergence of Weirs

For the measurement of irrigation water it is not the usual practice to install weirs where submergence is anticipated. However, changes in the regimen of the channel downstream may cause a weir to operate under submerged conditions. Submerged flow, at its best, is relatively unstable. Therefore, the results of the studies of submerged weirs are not in good agreement and it may be concluded that measurements made by a submerged weir should be considered as approximate only.[3,4] One solution is to remove the cause of submergence from the downstream channel if this is practicable.

Aeration of the Downstream Nappe of a Weir

One of the general conditions for accurate and consistent measurements by contracted weirs is that air circulates freely on all sides of the flow issuing from the weir notch. Such conditions ordinarily are not difficult to obtain. The weir bulkhead in irrigation structures is constructed of concrete in many instances. The use of metal weir blades which do not project a sufficient distance from the concrete, or an improper bevel of the concrete

Figure 8. Weir on which the crest has become appreciably rounded.

downstream from the blades can easily restrict the desired air circulation. The effect of this restriction of air is to increase the flow rate for a given head. The increase in discharge will depend on the degree of restriction of air and can be appreciable.

The problem is more pronounced when suppressed weirs are used. For standard suppressed rectangular weirs used in irrigation practice, the side-walls are generally carried straight through the structure. Thus, auxiliary means must be provided to supply air to the underside of the nappe. Unless adequate air is provided to this area to replace that carried away by the jet, a partial vacuum will be formed. The result is a lowering of the nappe and an increase in discharge over that obtained with adequate aeration. A condition of instability may also exist in which case erratic measurements would be obtained. Johnson(10) found that the discharge would be increased about 3-1/2 percent at 0.5-foot head and about 2 percent at 1.0-foot head when the pressure under the nappe was reduced only 0.8 inch of water below atmospheric. When the pressure was further reduced to 1.2 inches of water, below atmospheric, the increase in discharge was about 5 percent and 2-3/4 percent for heads of 0.5 and 1.0 foot, respectively. The size of vents adequate to relieve this negative pressure will depend on conditions at the weir. Both Johnson(10) and Howe(11) have developed solutions for calculating the size of vents. The important consideration is to design the vents of adequate proportions to relieve the low pressure insofar as is possible.

Other Factors Which Affect the Accuracy of Discharge Measurements over Weirs

There are factors, other than those covered separately above, which may cause errors in discharge measurements made with weirs. Many of them apply equally as well to other types of structures and devices.

Obstructions in the measuring section cause errors proportionate to the magnitude of such an obstruction. In irrigation systems floating detritus, weeds, moss, etc., may obstruct the water passage, Figure 9. Frequent and close inspection accompanied by remedial measures will relieve this condition.

Changes in viscosity and surface tension of the fluid are known to alter the flow coefficient. However, the effect of these two factors is considered negligible in irrigation systems where the flow media is water, and wide variations of temperature are not encountered and, further, provided that the restrictions on high and low heads over the weir are complied with.

At very low heads, flow over a weir may become quite unstable and errors and inconsistencies in the measurements will result. Because of viscous drag and the tendency of the nappe to adhere to weir crest there is general agreement among experimenters that heads of less than 0.2 foot will not produce reliable results when the usual discharge tables or formulas are used.

The results of many experiments on weirs show that the formulas developed for rectangular weirs do not hold when the head exceeds about one-third of the length. There are indications that the discharge formula for the Cippoletti weir, in lengths over 1 foot, is slightly in error at heads less than one-third the length.(12) Possibly the rule should be that the head should not exceed one-fourth of the length if errors are to be reduced to a minimum.

As previously stated the flow formulas for weirs have been developed empirically and are not necessarily susceptible to extrapolation. Most of the

Figure 9. Obstruction in weir notch—silt and vegetation in weir pool.

data have been derived for heads up to 2.0 feet. Although some data are available for higher heads, authorities generally agree that a 2.0-foot head should not be exceeded for any length weir if good quality results are desired.

It has been previously pointed out that the percentage of error in discharge resulting from a given error in measuring the head will decrease as the head increases. Therefore, the minimum error and, hence, the greater accuracy can be expected if the discharge occurs under the maximum head commensurate with the above limitations.

Careful visual inspections made at regular intervals will remove many of the sources of error mentioned above. These inspections should also disclose other sources of errors such as leaks around the measuring structure, through weir bulkheads, or from drains in the structure.

CONCLUSIONS

The charts, tables, and discussions presented in this paper are not intended to point out all the possible errors in all of the devices and structures used in measuring irrigation water. However, from the examples cited, the following conclusions may be drawn.

To obtain accurate measurements of irrigation water it is necessary to make a careful study for the selection of a proper device to fit the conditions pertaining at the site. Even with careful planning and selection of an excellent primary measuring device, it is probable that errors may be introduced into the measurements unless due care is exercised in fabrication, installation, operation, and proper maintenance of the devices or structures. The magnitude of these errors can be appreciable and the value of a well planned measuring program may be reduced considerably by failure to anticipate and remove the cause of the errors.

The possible errors cited are both negative and positive and may tend to cancel each other. However, more careful scrutiny shows, especially in the case of weirs, that the probability is that there is a predominance of negative errors. This means then that usually more water is being delivered than is apparent from the measurements.

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Paper 1363 September, 1957 IR 2

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DRAINAGE IN THE MISSISSIPPI RIVER VALLEY

Louis W. Herndon*
(Proc. Paper 1363)

SYNOPSIS

This paper explains the coordination between the Soil Conservation Service and the Corps of Engineers (United States Dept. of the Army) in drainage activities in the alluvial valley of the Mississippi River. The relation of flood control and drainage is discussed and the different elements of complete surface drainage systems are described.

INTRODUCTION

Since earliest recorded history the world's great river valleys have increasingly become the major sources of food and fiber production. Those civilizations and nations which have successfully developed their great river flood plains for agricultural production by protective and improvement works have prospered. Enormous public expenditures have been justified for their protection and development. The alluvial valley of the Mississippi River is one of these great world agricultural areas. No other area in the Nation can surpass its agricultural potential. Although production from this area is already tremendous it is only a fraction of its potential.

To realize the full productive capacity of this area requires the control of the very force which created it—repetitive flooding. The control of water so that sufficient, but not excessive, soil moisture is available for crop production, calling for drainage and irrigation developments, is important but logically must await initial flood control. The major flood-control works in the lower Mississippi Valley are now built and most of the flood-control work still needed is for improvements to basic systems and extension of areas

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protected. Irrigation is comparatively new to the lower Mississippi Valley, commonly referred to as the "Delta," but it is developing rapidly and, coupled with sound conservation farming, will prove the key to realization of the full potential of agricultural production. This paper will deal primarily with the intimately correlated roles of the Soil Conservation Service and Corps of Engineers in planning for drainage.

Drainage development in the Delta generally has followed provision of flood protection and is mostly localized in areas that are protected from frequent flooding.

The Boeuf and Tensas Rivers and Bayou Macon area has been selected for discussion in order to point out specifically some of the problems of drainage work in the Delta. Soils and climate in this area are similar to the rest of the Valley and crop response to proper agricultural water management here is comparable to other areas in the Valley.

Relation of Drainage to Flood Control

There is no sharp line of division between works of improvement for flood control and those for drainage. Flooding occurs when waterways have capacities too restricted to contain flows. Thus, overflow waters spread from confined, channelized flow to cover broad areas. To control this, the channelized flow is regulated by impoundment above with controlled release; or the in-bank capacity of the stream course is increased, either by excavation or by elevating the banks with levee systems; or by combinations of these measures.

Drainage is defined as improvement works installed for the purpose of lowering the water level in areas that under natural conditions of topography and under normal precipitation are too wet for sustained agricultural use. Drainage is accomplished by conducting runoff from rainfall which is in excess of the needs of the soil to an adequate in-bank, channelized flow, with removal rapid enough to prevent crop damage or cause increased costs of farming due to wetness of overlong duration. Sloping fields usually drain rapidly enough without man-made help. Coarser, sandier soils, even when flat, such as most of the so-called "front lands" along major streams, have sufficiently rapid natural rates of drainage. Water standing overlong on such lands is prima facie evidence that the problem is one of flood control, not drainage. Flat, heavy soils, however, have neither the topography nor soil texture to allow sufficiently rapid natural removal of rainfall. For fields composed of such soils a collection system, designed to draw water from crop rows, flat cropland, or pasture rapidly enough to prevent depression of crop yields, is the key element. This element may embody designation of row direction, land forming (a relatively recent development, consisting of one or more of the practices of land leveling, bedding, shaping, grading, or smoothing), and installation of row drains and small field drains.

Obviously, if water is concentrated in collection systems the drainage system must have also a disposal system, consisting of the lateral and main ditches, both group and on-farm, required to carry the water thus collected to outlets. And to complete the drainage system there must be a point of release of water from the disposal ditches of the system where the drainage outlet capacity is sufficient to prevent out-of-bank backwater or confine it to planned sumps. No drainage system is complete or fully effective without these three interrelated elements. To realize maximum drainage benefits

the complete system must be installed and maintained. Where this is done crop production is increased by: (1) Increasing soil bacterial action; (2) causing soils to warm up more quickly; and (3) increasing the root-zone depth for crops. Drainage systems are usually planned on the basis of removing a given depth of water from fields within a given length of time, depending upon the tolerance to standing water of the crops to be grown and the characteristics of the soil in regard to infiltration and runoff.

A great many alluvial areas are composed of intermingled areas of wetland soil inherently needing drainage, and coarser soils and benches of sloping land that are naturally well drained. Furthermore, sometimes there are adjoining hill lands that drain onto them. Drainage disposal systems of course must be designed to care for the runoff from these areas too, but obviously developing additional ditch capacity to care for water from these hill areas, either by excavation or levees, is not properly chargeable to the drainage of wetlands and must be considered as flood control.

Often there is an inseparable combination of both flood control and drainage benefit to the same areas of wetlands. This, in fact, is the most common situation in unleveed areas of the alluvial valley of the Mississippi. Here, the problem is one of determining the <u>net</u> system capacity for drainage and its cost, in comparison to the full <u>flood control and drainage</u> system capacity needs and cost. It may then be assumed that flood control and drainage benefits will accrue in proportion to the costs. Separation of benefits on some basis is necessary because Federal legislation requires different handling and financing of flood control and drainage developments. Some acceptable manner of measuring both costs and benefits of each type of development is an essential element in project planning under Federal laws. This is true, equally, for the Federal statutes under which both the Department of Agriculture and the Corps of Engineers operate in the field of flood control and agricultural water management.

It is possible for flood control and drainage installations to be at cross purposes. And, by the same token, flood control need not be complete (all flows contained in bank) to permit satisfactory drainage improvement. For instance, land adjoining a stream course, though subject to occasional flooding, may be highly productive and fully justify crop production. Some of the well-drained soils would not require drainage measures, while drainage installations for inherently wet soils could be fully justified. However, flood control measures involving both upstream impoundment and stream-course levee or floodway installation could contain all flows in bank, but through controlled release keep water levels high in the stream course through much of the crop-growing season. Such a flood-control plan might easily create high water tables and problems of drainage that hitherto did not exist. It has happened. This means, simply, that where both flood control and drainage problems exist in the same area, a combined system must be planned as a unit to be sure that both problems are alleviated for the ultimate good of the land to be benefited (in addition, of course, to due consideration of nonagricultural aspects).

The minimum degree of flood control ordinarily required, agriculturally, is that which will stimulate a level of agricultural development considered to be adequate in the area and which will allow economic developments, including drainage system works to be installed and maintained. There is some indication that owners of potentially productive alluvial lands in the Mississippi River valley will adequately develop lands which do not flood

during the cropping season more often than once in five years. This, however, is a generality which should be modified by more accurate appraisal in specific project study.

Drainage needs should be determined on the basis of the estimated net acreage that will require acceleration of water removal. Flood control needs, on the other hand, should be determined on the basis of works required to keep runoff from a certain expected frequency storm contained inbank, considering the entire drainage area involved. The difference in cost of the facility for providing drainage and the multi-purpose facility for providing both drainage and flood control is that which is clearly chargeable to flood control.

One thing is important to remember about drainage—although the on-farm collection element is the key element of the complete drainage system it should always be the last element, in time, to be installed. First should be provision of major outlets, and then disposal ditch systems. Thus, drainage development is in the serious position of requiring public expenditures first, with private group and individual installations following at a later date. This fact puts a premium on careful planning with adequate attention to realistic appraisal of the willingness and ability of landowners and operators to follow through with installation and maintenance of the small group and on-farm works. Sound public investment demands assurance of the necessary subsequent action of private interests to reap the benefits that justify the public assistance.

Early Drainage Developments in the Boeuf-Tensas-Macon Basin

Much of the early drainage work in the Boeuf-Tensas-Macon area was done for the purpose of reclaiming swampland for agricultural use. In this it was partially successful, as is evidenced by the fact that these reclamations continued on a large scale until the flood of 1927. However, these efforts to provide drainage have not always been financially successful. In the twenty years from 1907 to 1927 approximately $5,500,000 was spent on drainage works in the basin. For various reasons the benefits obtained were not always commensurate with the costs, and in many cases the bonds issued proved to be a serious financial burden to the landowners. After the disastrous flood of 1927, which inundated the entire basin, drainage activity was sharply reduced. A large number of the ditches were allowed to deteriorate, without maintenance. Landowners became discouraged because of serious damage to drainage works by the 1927 flood, because of the risks of more flooding, and because suitable outlets for their drainage ditches were lacking in many areas. Most of the improvements in Arkansas, for instance, emptied into the Boeuf River which had been enlarged only as far downstream as the Arkansas-Louisiana State line.

Many needed laterals were never constructed and the small surface drains in each field which are so vital for adequate drainage were seldom installed. Lack of proper maintenance resulted in a deterioration of many ditches which had been constructed.

These and other difficulties of an organizational or financial nature led to much dissatisfaction with early drainage developments.

Since the Flood Control Act of 1928 was passed by Congress the Corps of Engineers has completed works necessary for protection of the alluvial valley

from major river overflow, though large areas subject to backwater flooding still exist. Since the improvements to the Boeuf and Tensas Rivers and Bayou Macon and their tributaries were authorized by Congress a large percentage of the work needed to provide adequate major drainageways for this area has been accomplished also.

Work on these major drainage ditches is progressing upstream and, as areas develop agriculturally the requirements for major drains are reviewed and revised according to the needs.

In Louisiana the Department of Public Works, in a cooperative program with the parishes, has constructed systems of group lateral ditches which provide good outlets for nearly all of the area which can be served by major outlets that have been developed by the Corps of Engineers.

As a part of the national soil and water conservation program the Soil Conservation Service gives technical assistance to individual farmers and small groups of farmers in the establishment of drainage systems for their farms. This assistance is made available through locally organized and operated soil conservation districts. Also, as a part of the assistance furnished districts, the Soil Conservation Service, when requested to do so, makes surveys of the drainage situation in the district and advises the district supervisors on the overall drainage problems.

More recently, as a result of the Watershed Protection and Flood Prevention Act—Public Law 566, 83d Congress, as amended by Public Law 1018, 84th Congress, the Soil Conservation Service is authorized to give assistance to locally organized watershed groups in solving their agricultural water management problems as well as their problems of watershed protection and flood prevention.

This Federal and State assistance on flood control and drainage has been a big factor in the development of a more stable agriculture in the Delta. But for the success of all phases of flood control and drainage the local people must shoulder a big responsibility. Construction of all the on-farm drainage systems, installation of a large part of the group systems, and the maintenance of all the works is their job, individually and collectively. Theirs is a continuous job—the constant maintenance necessary and the revisions and reconstruction of farm drainage systems required as farming patterns and methods change.

As the major drainage ditches are extended into Arkansas, thereby providing outlets for some of the large drainage districts there, it is expected that local people will rehabilitate and expand their systems to take advantage of the outlets provided. Here is an area which in the next 5 - 10 years will see a tremendous growth in agriculture. The rehabilitation of group drainage works and construction of new facilities will require a lot of engineering assistance that should be obtained from engineers in private practice.

Review of Mississippi River and Tributaries Project

In order to consider and evaluate adequately all aspects of river basin development a number of agencies must correlate their activities and a lot of planning work is required. The review of the Mississippi River and Tributaries Project by the Corps of Engineers is a good example of the cooperation between various agencies on large-scale developments.

In May 1955 the President of the Mississippi River Commission requested

the U. S. Department of Agriculture to cooperate in the review of the Mississippi River and Tributaries Project. The Mississippi River Commission, through the Corps of Engineers, is conducting a review of the Mississippi River and Tributaries (MR&T) Project, authorized in the 1928 Flood Control Act and modified by subsequent acts, to determine the adequacy of the authorized project together with any modifications or additions that may be necessary. A part of the review consists of an economic evaluation of certain phases of the project by the Corps of Engineers. The Corps of Engineers asked the Department of Agriculture to supply certain basic agricultural information, and the Department agreed to conduct a study for the purpose of obtaining the information requested by the Corps.

This work is progressing on a schedule for project study established by the Corps. Governed by the desires of local interests, as set forth in a series of public hearings, the Corps of Engineers established a priority for study of the many individual projects involved in the overall review.

These individual projects are proposed Corps of Engineers works which are designed to meet certain needs within an area and to aid and complement the overall job of flood control and development in the Delta. The limits of the area for study are designated by the Corps of Engineers.

After receiving the designation of projects and certain maps and other pertinent information from the Corps of Engineers the Soil Conservation Service makes the investigations necessary to furnish the following information to the Corps:

1. A land classification survey of all land within the project area.
2. Estimation of crop response to flood protection and drainage. This is given in terms of expected yield increases and changes in cropping patterns due to drainage under flood-free conditions.
3. Estimation of the degree of farmer participation in the drainage program. Even though construction of the on-farm drainage system is the phase of the program that is ultimately the most profitable to the individual it is seldom that 100 percent participation is attained. Estimates are based on experience in similar areas.
4. Costs of crop production and land use conversion.
5. Estimates of cost of the group and on-farm drainage systems. These are based on limited investigations and studies of drainage costs in other areas of similar topography and soils.

Assistance in determination of the values of wood products is being given by the Forest Service, since flood control and drainage development undoubtedly would stimulate some clearing and conversion of woodlands to field and pasture crops. The Agricultural Research Service is furnishing information on agricultural economics and trends, and correlating certain Weather Bureau precipitation records and soils data in respect to periodicity and duration of both excessive soil moisture and soil moisture deficiency.

Since this is an area of important fish and wildlife values, the U. S. Fish and Wildlife Service is cooperating in the review. It is furnishing advice concerning the effect of project proposals on fish and wildlife habitat and making such recommendations as are deemed necessary in the interest of fish and wildlife.

The results of all of these studies are furnished to the Corps in a suitable

form. From the basic data thus furnished the Corps can complete its evaluation of flood-damage reduction to be effected by installation of the project. Finally, the various effects of the proposed project are weighed and an economic analysis is made. The recommendations for amended authorization will therefore be based on the results of a coordinated, inter-agency approach to a complicated development.

Project Development

Recent water-control improvements in the Boeuf-Tensas-Macon area have followed the sequence: (1) Flood control (levees), (2) major outlet installation, (3) group main ditches and laterals, and (4) on-farm drainage systems. This is proper and results have been good.

Following construction of a major outlet it usually takes several years for the installation of the group and on-farm systems. Initially, drainage runoff is much less than it is after considerable development of the watershed. This characteristic of drainage development indicates the desirability of following a procedure in design which initially provides only for the limited scale of development expected to be associated with early years of project life. Subsequently, and as the project area develops, the major drainage facility can be enlarged to serve a more extensive scale of development.

The drainage coefficient used in the design of the original improvements by the Corps of Engineers in the Boeuf-Tensas-Macon area was based on a scale of development which has now been exceeded. Major outlet ditches were designed to carry the runoff given by the formula:

$$Q = 35\ M^{0.83} \text{ where}$$

Q is the runoff in cubic feet per second and M is the drainage area in square miles.

When the ditches are reworked they will be given a larger capacity than they were originally designed for and more in line with the current stage of development.

Runoff formulas used by the Soil Conservation Service for design of on-farm systems range from $Q = 22.5\ M^{0.83}$ for riceland to $Q = 45\ M^{0.83}$ for row-crop land in Louisiana. Most group drainage systems in Louisiana are designed on $Q = 45\ M^{0.83}$. In Arkansas the runoff formula used for group drainage design is $Q = 40\ M^{0.83}$. The lower rate of runoff is used in Arkansas because of the difference in rainfall characteristics.

Some factors which enter into the selection of a drainage coefficient for a major outlet which the Corps of Engineers might construct are not applicable to selection of a drainage coefficient for small group or on-farm drainage systems. For small group or on-farm systems a coefficient is selected which will provide the required degree of drainage for the type of crops to be grown. This is also considered in selecting the coefficient for the major outlet, but the drainage area is so large that average conditions must be used. The major outlet may have large areas of its watershed for which it is not necessary to provide improved drainage. Then, too, it may be necessary to provide a greater degree of flood protection for certain areas in the watershed of the major outlet than is ordinarily required for agricultural drainage.

The approach used in design for the improvements of the Boeuf-Tensas-Macon area appears to be a logical solution to this problem.

The plan for drainage in the Camp Bayou watershed, Arkansas, illustrated in figure 1, includes the various phases of drainage development which have been discussed. The Boeuf River, improved by the Corps of Engineers, provides the outlet for a lateral ditch constructed by the Louisiana Department of Public Works in northeastern Morehouse Parish, Louisiana. This lateral extends to the Louisiana-Arkansas state line and is the outlet for the group drainage system of the Camp Bayou watershed. The Soil Conservation Service is assisting the Camp Bayou watershed group in preparation of drainage plans for the area under provisions of the Watershed Protection and Flood Prevention Act. After installation of the group drainage system the farmers in the watershed can construct their on-farm systems. Some features of on-farm drainage systems are illustrated in the typical layout of figure 2.

Maintenance

Continuous effective maintenance is essential for success of drainage work. For each part of the system a plan of maintenance and the means to execute it must be provided. This requires the designation of an organization to be responsible for the maintenance of each segment of the system. The organization must have legal and financial means to accomplish its job. As a prerequisite to construction of the major drainage ditches by the Corps of Engineers, the Police Juries of the parishes concerned in Louisiana have agreed to accept responsibility for maintenance. The Police Juries also are responsible for maintenance of ditches constructed cooperatively with the Louisiana Department of Public Works. In Arkansas, group drainage systems are usually constructed and maintained by drainage districts. The farmer is responsible for maintenance of his on-farm system in both states.

SUMMARY

The importance of the alluvial valley of the Mississippi River for agricultural production has been emphasized. Water-control measures necessary to attain the full potential of agricultural production from major river flood plains are—in the order in which they should be established—flood control, drainage, and irrigation. This paper has dealt primarily with drainage.

For proper drainage there should be no obstruction to the flow of runoff from excess rainfall from the most remote area to be drained to the outlet. The system to provide this drainage is divided into a collection system, a disposal system, and the outlet. The Soil Conservation Service and the Corps of Engineers have coordinated their efforts in the drainage activities in the Valley. The Soil Conservation Service gives technical assistance through local soil conservation districts to farmers and small groups of farmers in installing individual farm and small group drainage works. The Corps of Engineers has the responsibility for the major outlets. The Soil Conservation Service has furnished basic agricultural data to the Corps for planning purposes.

The intermediate or group systems of disposal ditches have been established under different types of organization. In Louisiana this is largely a project of the local parishes in cooperation with the Louisiana Department of Public Works. In Arkansas local drainage districts have done most of the existing work and will most likely handle the major part of additional work.

The Soil Conservation Service gives assistance in this field through provisions of the Watershed Protection and Flood Prevention Act.

It is essential to establish all elements of the drainage system to attain full benefits and the on-farm system is the key element, without which there are few benefits. Maintenance is necessary to all elements and must be provided for in the organization established to carry out the project and in the funds to be made available annually for operations.

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CAMP BAYOU WATERSHED
ASHLEY COUNTY SCD
ARKANSAS

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Figure 1

2-8-57 4-L-11200

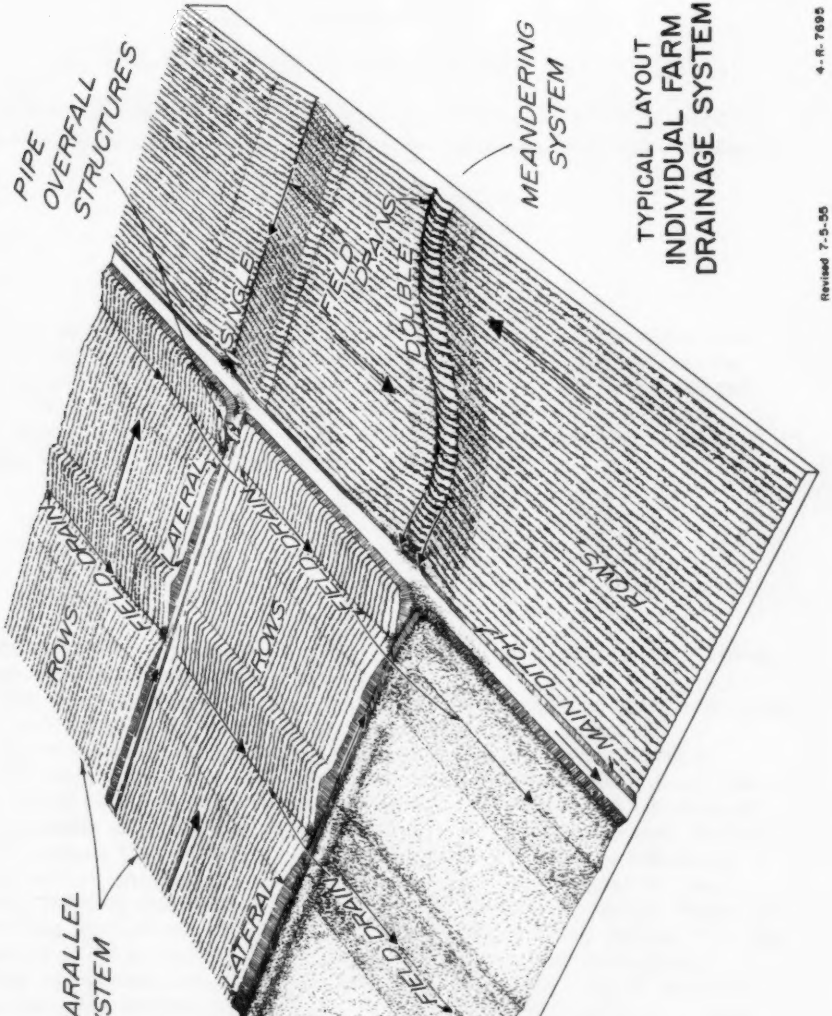

Figure 2

U. S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE

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WATER USE IN INDUSTRY*

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(Proc. Paper 1364)

The industrial use of water in the United States can readily come under the heading of big business. An estimate of the total daily industrial use varies from 20 to 25 billion gallons per day to over 80 billion gallons per day. The latter figure probably includes steam power condenser use. According to A.W.W.A. Task Group Report A4D1, the following amounts of water were used in the United States in 1950:

Use	B.G.D.	% of Total
Municipal (public)	14	7
Industrial (private)	81	40.5
Rural (farm use)	5	2.5
Irrigation	100	50

Even with steam generation omitted, the figure of 20 to 25 bgd is still 50 to 75 percent more than total municipal use.

Big water users are (1) electrical (power generation and equipment manufacture); (2) pulp and paper; (3) petroleum products; and (4) steel.

There may be considered four primary sources of water: surface, ground water, sea water, and reclaimed water. Water supply for an industry should be (1) abundant enough to take care of present and future needs; (2) available at quantities and pressures to meet all peak demands and provide adequate fire protection; (3) suitable quality for various industrial uses; and (4) constant quality if this is important for purposes desired. Water supplies and waste disposal only too frequently are not considered in delecting plant sites. Plant sites are often selected where maximum future quantity is not available. For example, wells may not be inexhaustible. In too many cases industries may be crowded together, causing very high demands on a distribution system. This is an important problem in arid regions such as the Southwest where many large water users are locating in the face of dwindling local water supplies and costly imported water.

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It may be that large water-using industries will ultimately be forced to locate in areas of plentiful water such as the Northwest and leave the more arid regions to those industries with lower water demands, or to practice water conservation. One observer has stated that water shortages are not due so much to lack of water as to lack of planning. Intakes from streams or large bodies of water may be too near sewers, which affects the quality of the water. It is not so much a problem of total quantity of water available to a water system as it is the inflexibility which makes rapid changes difficult in obtaining the amount of water available to any given locality. Once a main is in place, modification is a major operation which can be undertaken only after considerable delay.

Turneaure and Russell, in their book on water supply, stated that in 1890 average use throughout the Nation was 90 gallons per capita per day. This had risen by 1945 to 125 per capita per day or a 40 percent increase. An average use in five major cities (Philadelphia, Detroit, Providence, Cleveland, New York) gave a 2 percent yearly increase in a decade between 1933 and 1942 and a 4 percent yearly increase in the 5-year period 1943-48. Industrial use has increased approximately 40 percent in the past decade. This increase varies greatly with the industry. As an example, 5 percent to 6 percent of all industries use 80 percent of the industrial water use whereas 67 percent of the industries will use less than 10 percent of total industrial water use.

Use of water varies by industry and by time. Many industries will only have 8-hour operation whereas others, including many big water users such as steel mills, will have 24-hour operation. Peak flows will often exceed 200% of average flow. For example, in canneries where retort cooling is necessary, these high demands may affect the location of the industry with respect to the layout of the municipal distribution system if the industry expects to take water from a public system. Frequently, distribution mains that are ample for all ordinary uses including fire, will be totally inadequate for these high special industrial demands. This is especially true where pumps taking suction directly from the mains are used. Their use should never be permitted. Efforts, of course, should be made to locate big water users in areas of large mains. If some users have their demands on off-peak hours, they may be integrated in other areas.

Many large users of water—for example, paper mills, chemical works, textiles, etc.—frequently have treatment plants equivalent to that necessary for a city of several hundred thousand. In addition, these treatment plants may require very expensive special equipment because of the high grade of effluent required. As an example, one rayon industry with a water treatment plant of 16 mgd capacity has all rubber-lined piping and all nickel valves. In another case, a chemical plant with boilers operating at 1250 pounds psi pressure, a 6 mgd demineralization plant costing $1,500,000 is in use.

Quantity

Industrial use of water divides itself very readily into three major categories: quantity, quality, and re-use. In considering the quantity, exact figures are often very difficult to obtain. Water use by industry may be given six general classifications; viz, processing, power generation,

sanitary, fire, and miscellaneous, including the growing demand for air conditioning in electronic plants, etc. This air conditioning requirement is not so much for the comfort of the employees as for the exacting conditions necessary for the production of a satisfactory product. The varying proportions of the amount used for these different purposes depends not only on the type of industry but also on the individual industry itself. Variations in the amount of cooling water, for instance, in any given industrial plant will vary as much as 100 or 200 percent more, depending on type of equipment in use and waste disposal, and whether there is re-use of water. An industry adjacent to a large body of water may dispose of cooling and other waters back to the lake or stream, whereas as inland industry may salvage and re-use these same wastes, thereby reducing the consumptive use to make up water only.

Exact figures for any given type of industry are not often broken down into specific uses and as far as can readily be obtained, such figures will be largely approximate. There are a number in instances where a general estimate of entire water use for a given industry may be obtained. A very comprehensive table is given in Eskel Nordell's excellent book on water treatment. A partial listing of average water use in some typical industries is given below:

Rayon - 16 gallons per hundred pounds
Wool scouring - 126 gallons per hundred pounds
Oil refining - 770 gallons per barrel
Steel - 65,000 gallons per ton
Paper - 39,000 gallons per ton
Soda pulp - 85,000 gallons per ton
Alcohol - 100 gallons per gallon of alcohol
Beer - 470 gallons per barrel
Electric power - 80 gallons per kilowatt hour
Boiler use, general - 4 gallons per boiler horsepower power
Condenser cooling - 2-1/2 to 7 gallons per pound of steam
Dairies - 100 gallons per can (44 quarts) of milk

In a large number of industries the chief water uses are boiler, cooling, and process or general use. Cooling water use, in general, exceeds all other uses. A large power plant may require 500,000 gallons per minute for condenser use. Large quantities of water may also be used for cooling a product such as cooling of cans in retorts. There is very little data in this regard for estimates. Process water in general is a small portion of the total. A canning company using more than 2 bgd of water per year estimates only 6 percent is used for the food product. Somewhat more may be used for washing, soaking, and can washing. Fire protection and flushing of lines average approximately 12 percent of the total water used.

Water requirements for fire protection depend upon fire underwriters' requirements, which in turn depend upon the type of industry, type of buildings, etc. If an industry depends on municipal water supply, it may be that they must pay part of the cost of providing sufficiently large mains to take care of the fire flow or the industry will have to provide for its own fire supply. The quantities required for fire protection are in the nature of a standby demand that may be imposed on the peak industrial load. If the standby capacity is not built into the distribution system, on site storage and pumping facilities may be required. This may be done by either taking

directly from a large natural body of water or providing a reservoir of sufficient capacity to give maximum fire flow for a given period—say 45 minutes.

Quality

Water quality for industry should be of constant nature. This is especially true for waters used in processing. Sudden changes in quality may cause great difficulty, expecially where water may be treated. Examples of this occurred in Los Angeles and Long Beach a number of years ago where water was used for ice making. Local waters were moderately mineralized with between 200 and 300 ppm total dissolved solids and were of the sodium carbonate class. Imported waters were highly mineralized—700 ppm total dissolved solids—and of the sodium chloride-sodium sulfate type. In certain areas the imported supply entered the local supply through regulators. This permitted slugs of the highly mineralized water to enter the system at unpredicted times. Ice plants in these areas frequently found extreme hourly variations of water quality. These variations in quality caused a large number of white butts and milky cakes, resulting in a poor grade of ice. An oil refinery also using local waters of the sodium carbonate class would occasionally get slugs of imported water of the sodium chloride-sodium sulfate class. This radically affected their demineralizers for boiler water use and completely upset their processes. Since the operation of the water system made it impossible to change this procedure, both the ice companies and the oil company were forced to put in storage tanks, periodically filling them, analyzing the water, and making their adjustments in treatment to conform to the type of water in the tank. Needless to say, this puts a rather heavy burden on the industry both for operation as well as capital outlay for additional equipment.

Water quality requirements will vary greatly for different industries and it is obviously difficult for a municipal supply to meet requirements of all industries for special purposes. In general, municipal supplies conform to the United States Public Health Service standards of physical, chemical, and bacteriological quality of water. If the industry desires refinements beyond these requirements, they should expect to do their own treating, develop their own supply, or both. An example of this would be the clear water of extremely low turbidity required by the film industry.

Process waters may vary greatly in requirements. For example, a water high in sulfates or calcium is generally perfectly satisfactory for, and even to be desired in, a fermentation process. It is quite unsatisfactory, however, for canning certain foods such as peas, beans, lentils, etc. In the latter case, the total hardness combines with the protein, making the product tougher and less desirable. In the case of canning beets, the oxalic acid in the beets forms calcium oxalate with hard water and produces an undesirable whitish deposit on the product. Zero hard water is preferable for cake and cracker baking. Wash water should preferably be of zero hardness and may even be demineralized if the water has high total solids. Otherwise, spots on cans and glassware may be caused.

Seasonal changes in water quality may not be too objectionable and quite frequently occur in municipal supplies using a surface source of water. These will generally not cause particular difficulty in adjustments since the industry will have opportunity to check the quality periodically. When a water utility proposes to vary its quality or source in future operations,

industry may be seriously affected and should be informed if possible. Steam power generating plants have rigid quality requirements and provisions should generally be installed for treatment of all types of water quality combinations anticipated. If water of a constant quality is desired, the industry should either be located in an area where the quality will be constant or they should develop their own well supply since, in general, ground water supplies are of fairly constant quality as compared to surface supplies.

There are many types of treatment processes. The type of process to be used depends on the quality of water to be treated and the use desired. Each of the four major water uses in an industry may pose a specific requirement for quality and temperature within a given class of industry. This requirement may vary from plant to plant. For example, cooling water in a volatile solvent distillation plant may have to be of much lower temperature than condenser cooling water in a steam power generating plant.

Methods of water treatment may be broken down into seven classes: zeolite process; demineralization; hydrogen-cation exchange; cold lime soda; hot lime soda; filtration; iron and manganese removal. The following are some examples of the application of these treatment processes to specific water uses. Boiler water should be quite soft and with very low silica content, particularly in the case of high pressure boilers. Boiler feed waters may be softened by the hot lime soda, a lime zeolite combination, zeolite alone, or hydrogen-cation exchange process. Silica removal, when required, is usually affected by dolomitic lime or activated magnesia in combination with either hot lime soda or cold lime process.

Cooling water may often be used with very little treatment for condensers, furnaces and engines, although turbidity, hardness, and organic matter should not be excessive. For some special uses, particularly in connection with food products, a higher type of water must be used. For plating operations and the cleaning of metals in connection therewith, softened or even demineralized water is preferable. This prevents waste of detergents and the formation of deposits on the surfaces from the salts in the water. General purpose water for such domestic uses as drinking, cooking, and lavatory use should be clear, colorless, tasteless, odorless, free from iron and manganese, and of approved bacteriological quality. In the food industry, water used for the product should, of course, meet all requirements of the United States Public Health Service for drinking water, particularly bacteriological quality. Water for can and bottle washing is preferably softened. Iron and manganese are objectionable in that they may cause yellowish or brownish stains, may affect the product adversely as to taste, and frequently will give rise to growths of certain types of organisms which will cause objectionable tastes and odors in the water and in the product. Iron and manganese may be removed by aeration, lime treatment, settling and filtration, or sometimes base exchangers such as zeolite or demineralization. Surface waters, in general, will require sedimentation, coagulation, settling, filtration, and even chlorination. In some food plants where the flavor of the product is of great importance it might be adversely affected by chlorine or other tastes or odors in the water. In such instances, ultraviolet sterilizers and ozonizers may be used for sterilization in place of chlorine, or activated carbon filters may be used to remove chlorine or other tastes and odors. The use of an ultra-violet sterilizer is predicated

upon a very clear water. A turbid water will frequently be improperly sterilized due to the shielding action of the particles of turbidity on the bacteria present.

In the case of breweries, their use of water may be in four categories; brew water; cooling water; water for bottle washing, keg washing, and pasteurizing; and boiler feed water. Brew water, of course, should be clear, colorless, odorless, tasteless and free from iron and manganese, and bacteriologically acceptable. For making the light-colored pilsner type of beer, a low bicarbonate and high sulfate content is desirable. Brew water for such beers, if high in carbonates, is usually lime-treated to reduce its content. If low in sulfate hardness, calcium sulfate is frequently added with the lime. For darker types of beers, the bicarbonate content is usually not significant.

In the case of the chemical industry, since this is an industry of a wide variety of products, standard requirements for water are difficult to enumerate and are largely dependent, of course, upon the type of chemical produced. Most large chemical industries will require boiler feed water and cooling water, the quality requirements of which have already been considered. The amounts of cooling water required in various chemical plants will cover an exceptionally wide range from a fraction of the total water consumption to over 90 percent in many cases. If cooling water is once used and then run to waste, frequently no treatment will be required. At times, however, particularly in the case of sea water, intermittent chlorination may be advisable for the control of slime growths. If the cooling water is reused by circulation through cooling towers or spray pods, treatment for prevention of scale and organic growths will usually be required. Scale may be controlled by the various softening processes such as cold lime, zeolite, etc. Organic growths may be controlled by chlorination, copper sulfate treatment, or treatment with some of the newer organic algicides such as Phygon, Delrod, etc. Process waters in the chemical industry may be used for solution, purification by crystallization, washing, gas scrubbing, etc. Requirements for process waters, of course, will vary widely. In general, however, they should be clear, colorless, odorless and free from iron, manganese, carbon dioxide and organic growths. Whether further treatment of the process water is required will depend largely upon the chemicals being produced, the tolerances for various impurities and the composition of the process water. If the mineral content of the process water is not unduly high and a commercial grade of chemical is being produced with liberal impurity tolerances, the process water may need no further treatment. If the product is of a chemically pure class or for drugs and pharmaceuticals, distilled or demineralized water may be required.

In the case of textiles, water should be free from turbidity, color, iron, manganese hardness, organic growths, and non-corrosive. Turbidity may be removed, of course, by coagulation and filtration. Color may also be removed in the same manner or with additional treatment by chlorination or ozonation. Iron and manganese are extremely objectionable because of their staining properties and in some processes because of their catalytic effects on various dyes. For high grade products, iron should not exceed 0.1 ppm; manganese, .05 ppm. These may be removed by such processes as aeration, pH adjustment, settling and filtration. Hardness is objectionable for so many of the textile operations that usually the entire plant supply will be softened, even when the initial hardness content is quite low.

In the cotton process, hardness forms insoluble soap curds and adherent deposits which stick to the material, preventing even penetration of the dye, wasting dye stuff and producing an inferior product. The same may be said of linen processing. Hard water increases breakage in silk reeling. Zeolite softened water produces a better product in rayon manufacture. Wool processing and dyeing will also require softened water. Corrosion inhibition, which may sometimes be necessary in softened water, may be aided by the use of sodium silicate treatment. In any case where water is softened to zero hardness, special piping may frequently be necessary to reduce corrosion. If silicate is added as a corrosion inhibitor, boiler feed must be taken off before the point of adding the silicate.

Water for atomic energy processes, in general, will parallel those of other industries. Some unique requirements exist, however, and should be emphasized. Certain special requirements which may exceed those in many other industries are needed. These special needs are (1) extremely rigid standards, especially those defining permissible concentrations of certain dissolved solids; (2) low temperature and high rates of consumption of water used for heat exchange; (3) extraordinary provision for uninterrupted water service. The rigidity of the water quality standards is made necessary by the fact that it could be highly objectionable if water used for heat exchange in a water cooled reactor contained dissolved salts of an element that, when irradiated by neutrons, has a long half-life. Costly treatment may be required to reduce the radioactivity of such water to a safe level when such water is released to the ground, sewer, or other waterway. In certain processes, completely de-ionized water may be required. The high temperatures from nuclear fission require effective functioning of heat exchangers at all times with no possibility of interruption of service that might result in costly damage to an important facility.

Reclaiming and Re-using Water

The quantities of water used by industry may often be very radically reduced by proper re-use. No better example of this may be found than in the steel industry. The national average of water use is 65,000 gallons per ton of steel produced. At the Fontana, California plant of the Kaiser Steel Company this quantity has been reduced to 1,000 gallons per ton of steel due to the very efficient waste treatment and re-use system developed at this plant. Not only will the re-use of water effect a saving of the total quantity of water used, but often the re-used water may be obtained at considerably less cost than the original supply. This is very well illustrated by Nichols in the case of a refinery at Amarillo, Texas. The city supply costs approximately 13.6 cents per thousand gallon. The cost of reclaimed sewage used by the refinery varies from 13.3 cents to 6.9 cents per thousand gallon, if the quantity treated varies from 1.5 mgd to 4.5 mgd. This price for reclaimed water includes an operating cost of one cent per thousand gallon for chlorination.

Nichols condludes "The use of reclaimed sewage by industry has definite possibilities in the Southwest as water conservation."

The use of reclaimed domestic sewage has, of course, been given considerable publicity in the operation of the Bethlehem Steel Corporation outside of Baltimore. In this instance, practically the entire sewage flow of the City of Baltimore is used by the steel plant at a cost considerably less than they could develop and treat their own supply from the river.

In any consideration of re-use of sewage, the increment of increase in total solids above that of the natural water supply must be given consideration. This will vary from city to city, depending on the type of city, i.e., whether largely residential, largely industrial, etc., and the original quality of the raw water supply. In general, however, total dissolved solids may be expected to increase by 50% to 100%. Chlorides may increase from 5 to 15 times the original raw water content. Sulfates will increase 1 1/2 to 3 times and silica 15% to 80%.

In many industries water requirements can be met by use of salt water or brackish water where available. Examples may be condenser cooling, fluming of uncut fish in canneries, wash-down purposes, toilet flushing, and many similar uses. To safely use such waters, equipment must be made of corrosion-resistant material. No plant in a water critical area should be designed to operate only on fresh water where saline water is available. Obvious as this is, many industries in coastal areas use no salt water, the principal reason being the high cost of corrosion-resistant materials. A thorough study of this problem should be made, however, as large savings in use of such low-grade waters may be made in spite of the expenditure for corrosion-resisting equipment. In any situation where unpotable supplies are used along with potable supply, great care must be exercised to see that the various water lines are properly marked so that there will be no danger of cross- or inter-connection between the two supplies. State or local health departments generally have very strict regulations covering such conditions, but only too often inter-connections are made with resultant damage, injury, or even death to employees.

In further considering costs of water for industrial use, Watson gives the cost of water to industry from municipal sources as 25.6 to 12.9 cents per thousand gallon, or a weighted average of 13.6 cents. This is based on a study of 124 industrial plants with consumption varying from less than 50,000 gallons per day to over 1,000,000 gallons per day. Considering private water sources of the industry itself (based on a study of sixteen plants with less than 50,000 to over one million gallons per day) the cost varied from 7.8 to 3.0 cents per thousand gallon with a weighted average of 3.3. The General Electric Company, considering plant-developed wells, produced water at 3.7 cents per thousand gallons. Water from surface supplies with chlorination as only treatment cost 3.9 cents per thousand gallons. If full treatment, including filtration, is used the cost rose to 8.9 per thousand gallon. Watson, in his very excellent article on "Need for Water Management Program in Industry" concludes: (1) that the water resources of the Nation must be preserved and protected because they are of vital importance to the future development of industrial, municipal, and individual enterprise; (2) industry should adopt a management program which includes the selection of the best source of supply, control of water throughout all the manufacturing process, and delivery of an effluent that does not destroy the water resources of the area and others; (3) because fixed quantities of water are required in the operation of a large industry, the cost is of great importance; (4) major industrial water consumers should adopt a water management program; (5) the fundamental steps which a plant should incorporate into its management program are: a) evaluation of current concumption cost conditions, b) practice of conservation throughout the plant, c) employment of recirculation and reuse wherever justified; and (6) water management pays cost reduction dividends, according to experience of the General Electric Company.

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DISCUSSION
(Proc. Paper 1377)

Discussion of
"PUMPING REQUIREMENTS FOR LEVEED AGRICULTURE AREAS"

by Gordon R. Williams, M. ASCE[1]
(Proc. Paper 1236)

H. W. ADAMS.—The problem of selecting the proper capacity for drainage pumping stations for rural leveed areas has been thoroughly analyzed in this paper. The basic approach is essentially the same as for the design of leveed urban areas, particularly if storage basins can be utilized in the latter areas. The writer presented similar hydrologic criteria for pumping capacities for urban areas some years ago.[2]

The economic analysis presented in the present paper is not applicable to urban areas, because even infrequent interior flooding of leveed urban areas cannot be tolerated. The costs and benefits of the pumping stations must be combined with the total costs and benefits of the entire levee project. If permitted, interior flooding and resulting damage in urban areas merely varies the cause of the flood damage and nullified some of the benefits of building the levee. On the other hand, the design of interior drainage facilities for leveed agricultural areas can be treated on the calculated risk or point of diminishing returns basis, for the reason that the farmer is more philosophical than the urban property owner about suffering losses resulting from the vagaries of nature.

The development of the design storm and runoff is on a very conservative basis for the following reasons:

1. The design rainfalls are assumed to occur simultaneously with flood stages in the river.
2. Rainfall depths of equal frequency up to durations of 6 days are assumed to occur in the same storm.
3. The runoff rates and volumes are assumed to have the same frequency as that assigned to the synthetic design storm.

To overcome the first objection, it would be necessary to have a long record of river stages and coincident rainfalls in order to develop the frequency of coincident events. Studies of this type were undertaken by the Corps of Engineers for the design of pumping stations for urban centers in the Ohio River Valley and at other places. Even if the records were available, such detailed and laborious statistical studies might not be justified for the protection of rural areas.

The development of a design storm from rainfalls of the same frequency for all durations results in a storm which is probably much rarer than the assigned frequency. It is much easier to criticize this technique (which the writer has often used) than to suggest an alternative. The following are suggested methods for study:

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1. Prof., Hydr. Eng., Massachusetts Inst. of Technology, Cambridge, Mass.
2. "Drainage of Leveed Areas in Mountainous Valleys" by Gordon R. Williams, Trans. ASCE, Vol. 108, 1943, pp. 83-114.

1. If sufficient records exist, use actual storm patterns that have occurred over periods of one to six days or more. The runoff from the storms could be computed by use of the unit-hydrograph after assuming average rates of infiltration. If sufficient storms were analyzed, approximate frequencies could be assigned to the computed runoff rates and volumes.

2. Make a study of runoff versus duration for some similar stream gaging station in the area and reduce it to a unit-runoff (inches) basis. The computed depths could be related to frequency.

The design storm used in the paper has a maximum duration of six days. It is possible that a longer rainfall period might prove critical if the pumping capacity is relatively small, such as 0.50 inch per day, and the storage is equivalent to only 2 to 3 inches of runoff. For example, if a high rate of initial runoff fills the storage area and then the rate of rainfall-excess continues at a rate slightly in excess of the pumping capacity for intermittent periods of more than 6 days, considerable damage may result. This was the problem in the past with flood control reservoirs which were designed to modify a severe isolated flood but were filled to a point above the spillway crest by a series of small floods from which the average inflow was only moderately in excess of the non-damaging outflow capacity.

If the pumping operation is to take place over 10 or more days, the required pumping and storage capacities might be computed from a mass curve of rainfall-excess computed from actual rainfall records of durations up to 30 days without introducing the unit-hydrograph. The required capacities of pumping station and storage basin would still be obtained by the use of a tangent line, the slope of which would be equivalent to the rate of pumping. With flat lands, appreciable storage and long duration of flood stages the problem resolves itself into one of ordinary land drainage. Experience on the Upper Mississippi River indicated that pumping capacities should be not less than 0.50 inch per day for satisfactory operation.[3]

Too often in hydraulic engineering design a very approximate frequency approach is adopted in the hope of avoiding overdesign but operating experience later proves that there was unintentional overdesign. The best test of a design is a chronological one in which an actual record of runoff (or runoff estimated from rainfall) in the same or similar locality is routed through the pumping facility. Rational designs must wait until more data are collected on operating experiences with completed projects as was done in Reference 3. It is unfortunate that Federal projects are usually transferred to local agencies for operation, and the latter usually have no interest in checking the adequacy of the design unless real difficulties arise. The result is that possible overdesign methods go unquestioned as long as no interior floodings occur.

3. "Cost of Pumping for Drainage in the Upper Mississippi Valley" by John G. Sutton, U. S. Dept. of Agriculture Tech. Bull. No. 327, 1932.

Discussion of
"SAFE YIELD IN GROUND-WATER DEVELOPMENT, REALITY OR ILLUSION?"

by R. G. Kazmann
(Proc. Paper 1103)

R. G. KAZMANN.[1]—The writer is indebted to the discussors for the opportunity to clarify some of the issues presented in the paper.

As an example of the semantic confusion associated with the word, Mr. Kramsky's understanding of the term "safe yield" is manifestly at variance with the meaning of the term expressed by Messrs. McGuinness and Ferris.

Mr. Kramsky's explanation of "safe yield," as applied to the Raymond Basin, brings to mind the system of oil-production proration popularly known as "allowables." Each oil field is allowed to produce a certain quantity of oil each month by governmental authority. Analogously, the water-master permits the ground-water users of the Raymond Basin to extract a predetermined quantity of water each year. In effect each user, and the entire basin, has a Permissable Annual Ground-water Offtake (PAGO) which is determined by the Court acting through a watermaster. Were the Raymond Basin to depend upon artificial recharge based upon a water-right (instead of accidental recharge, without a water right) it would meet the requirements for a Water District in a perennial-yield area as outlined in the paper. It may be noted that if the word "PAGO" is substituted for "safe yield" throughout Mr. Kramsky's discussion, the sense of the discussion is unchanged. Mr. Kramsky's discussion has greatly contributed to clarification of the situation, since he has illustrated, by specific example, the general principles contained in the paper.

Since all that is needed is a minor change in terminology, it should be evident that the quantity of water involved in PAGO (Mr. Kramsky's "safe yield") is a term with an exact meaning. The magnitude of the PAGO in any situation is determined in part by law, in part by conditions in the vicinity of each well, in part by conditions in the entire aquifer, in part by custom, and in part by recognized prior right. It is not determined solely by physical circumstances susceptible of measurement by steel tape or current-meter. It cannot be determined by a purely scientific investigation of physical conditions. PAGO is definable for legislative purposes and, after the "ground rules" within a particular district have been established, each well-owner will know exactly how much water he may legally pump during certain predetermined periods of time. There is nothing inherently vague or confusing in the concept.

It should be noted that the complexity and breadth of meaning implicit in "PAGO" is not implied by "safe yield" as used by Mr. McGuinness. His discussion clearly brings out the Survey's position an "safe yield," to wit: "Each ground water reservoir has a <u>sustained yield</u> to which it can be developed for human use." (Emphasis by C. L. McGuinness.) This is precisely

1. Consulting Engineer, Stuttgart, Arkansas.

the issue: the paper was written to call attention to the fact that it is this particular assumption which is incompatible with the present doctrine of appropriation (besides being incorrect). The writer's paper states ". . . the concept of a "safe yield" (Mr. McGuinness' "sustained yield") of aquifers, independent of considerations of regional hydrology, cannot be reconciled with the doctrine of appropriation. All water pumped from the ground must be replaced by water coming from the land surface if a perennial water supply is to be obtained from the ground, laws notwithstanding." Manifestly, if all the surface water is appropriated, ground-water offtakes cannot be sustained indefinitely, since no net recharge will be permitted.

A reinforcement of the writer's thesis occurred on November 29, 1956, when the State Engineer of New Mexico issued an order stating that, (1), the surface and ground waters of the Rio Grande Basin are interrelated parts of a single supply and that, (2), any ground water withdrawal ultimately results in an equivalent diminution of surface water flows thus materially decreasing the already fully appropriated surface water supply. This order, based on detailed study of a specific situation, would seem to effectively dispose of the generality that each ground-water reservoir independently "has a sustained yield to which it can be developed for human use." Apparently the position of the Geological Survey on "safe yield," as reported by Mr. McGuinness, stands in need of reexamination by that Agency.

Mr. Ferris states that the writer has used "safe yield" and "perennial yield" indiscriminately. Throughout the paper the words "safe yield" have appeared within quotation marks simply because the writer does not know what the term means. The term "perennial yield," as used by the writer, means the hydrogeologically assured steady-state output of a ground-water supply. The Raymond basin would have a perennial yield of 30,800 acre-feet a year in the strictest sense of the term if the Court were able to legally assure the Basin of this quantity of water for recharging the basin. It presently has a PAGO of 30,800 acre feet a year, based on accidental recharge, not by way of legal right. It is true, however that a portion of this PAGO results from the seepage of purchased water, so that in this instance some portion of the PAGO is legally assured.

Mr. Ferris quotes the writer's 1948 paper as evidence that "safe yield" can be expressed quantitatively. The writer's prior statements would seem to have no bearing on the correctness of the thesis presented in the present paper. Nevertheless, to clear up the matter, the quotation from the writer's 1948 paper did not refer to "safe yield," as Mr. Ferris implies. It referred to the prediction of potential offtakes from ground-water systems replenished by the proven infiltration of surface water. Nor does the writer consider the quotation optimistic in view of his own experience since 1948. This is reinforced by Mr. Kramsky's report on the regulation of offtake in the Raymond Basin—the answers obtained there, for an entire basin, are quantitative, reliable, and legally enforceable. This would seem to demonstrate that "reliable quantitative answers" are available from the science of ground-water hydrology.

Finally, in reply to Mr. Ferris, regulatory agencies should administer water law, not create it. The law itself should be as specific as possible and should define terms, so that the legislators and the public will know what the principles and objectives are. The improved technical "know-how" found in the staffs of State Agencies, noted by Mr. Ferris with approval, has also been favorable noted by the writer. But this improvement in technique is no

substitute for clear-cut public policy embodied in law.

The writer appreciates the opportunity afforded to strengthen the paper, and hopes that improvements in the water law, as well as in engineering technique, will be facilitated by a meeting of minds between scientific agencies and engineers in private and public practice.

Paper 1379 September, 1957 IR 2

Journal of the
IRRIGATION AND DRAINAGE DIVISION
Proceedings of the American Society of Civil Engineers

CLIMATIC INFLUENCES ON CROP WATER REQUIREMENTS[a]

Thomas C. Skinner*
(Proc. Paper 1379)

SYNOPSIS

Until recent years scientists knew very little about crop water requirements, rates at which different plants used water and particularly how climate influenced the water use rates of plants.

In the past decade, however, water requirements of plants as influenced by climate have received considerable study and experimentation. This paper is devoted, primarily, to a discussion of the work that has been done in this field to date with particular emphasis on work done at the University of Florida.

Until very recent years, irrigation was a word foreign to the vocabulary of farmers in the so-called humid area East of the Mississippi River. Irrigation was essential in the arid West, but to the farmer in the South and East where the average annual rainfall ranges from 40 to 60 inches, the use of additional water on crops was unthinkable—unthinkable that is, to all except a few farsighted individuals who early recognized the fact that total annual rainfall was no sure indication of adequate moisture in the soil at the time it was critically needed by a growing plant.

Irrigation engineers have recognized for many years that a proper soil and moisture relationship should exist for optimum plant growth. Until recent years, however, little reliable data have been accumulated which can be used to determine crop-water requirements. The lack of reliable data here, has made an accurate determination of when to irrigate and how much water to apply per irrigation most difficult if not impossible. In the past decade much work has been done in this field and various formulas or methods for making this determination have been advanced.

Note: Discussion open until February 1, 1958. Paper 1379 is part of the copyrighted Journal of the Irrigation and Drainage Division of the American Society of Civil Engineers, Vol. 83, No. IR 2, September, 1957.

a. Presented at a meeting of the ASCE in Jackson, Miss., February, 1957.

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In 1948, Thornthwaite[1] presented a formula which is being used to compute the water requirements of crops. At about the same time Penman[2] developed a formula based upon energy relations, for use in determining crop-water use.

Early in 1952, crop-water requirement studies were begun in Florida by McCloud.[3] A modified Thornthwaite type evapotranspirometer was used to determine measured water use. Calculated water-use rates were computed according to the formulas of Blaney and Criddle,[4] Tabor,[5] and Thornthwaite, and compared to measured use.

It was found that, at Gainesville, water-use was underestimated by all the formulas when the mean temperature was above 70°F. Blaney and Criddles' unadjusted linear formula was only in accord with measured values at temperature of around 65°F. At lower temperatures predicted values are high and at higher temperatures predicted values are low compared to measured values. Tabor's exponential relationship gives a better fit, but does not increase fast enough at high temperatures. Thornthwaite's unadjusted curve more closely parallels the measured water use in the range 50-75°F. but fails to provide for the sharp increase above 75°F.

In an attempt to mathematically express the relationship between measured water use at Gainesville and mean temperature, McCloud[3] developed an emperical formula then fitted an exponential curve, by least square, to the data. The formula:

Potential Daily Water-Use = $KW^{T - 32}$ was found to fit the data best when $K = .01$, $W = 1.07$, and T = mean temperature in °F. The measured use rate increased rapidly with high temperatures. Temperatures in the high region (around 80°F.) produced water use rates which were slightly higher than expected on an available energy basis, however, over most of the temperature range, measured values are within the limits of available energy. The above formula was used to compute a predicted weekly water use value. A high corrolation was found between those predicted and the measured water use rates.

This water-use rate, or evapotranspiration of plants, is an important aspect of the crop water balance. Assuming that it is a reasonable estimate of the cropwater use rate, by using a simple bookkeeping procedure, water balance can be computed from precipitation, evapotranspiration, run-off, and soil storage data. McCloud refers to this relationship as the Agrohydrologic Balance. This relationship is expressed diagramatically in figure 1.

First the plant-soil water storage reservoir must be determined. This is a function of the plants rooting depth and the available water-holding capacity of the soil. Thus, for a particular plant-soil combination there is a maximum water storage. Precipitation replenishes this plant water supply while

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1. Thornthwaite, C. W. An approach toward a rational classification of climate. Geographical Review 38: 55-94. 1948.
2. Penman, H. L. Natural evaporation from open water, bare soil, and grass. Proceedings Royal Society (London) A 193: 120-145. 1948.
3. McCloud, D. E., Unpublished Data, Florida Agricultural Experiment Stations.
4. Blaney, Harry F. and Criddle, Wayne D. Determining water requirements in irrigated areas from climatological and irrigation data. United States Department of Agriculture, Soil Conservation Service. 1950.
5. Tabor, Paul. Standard rainfall, Proc. Amer. Soc. Horticultural Science. Pp. 594-598. 1931.

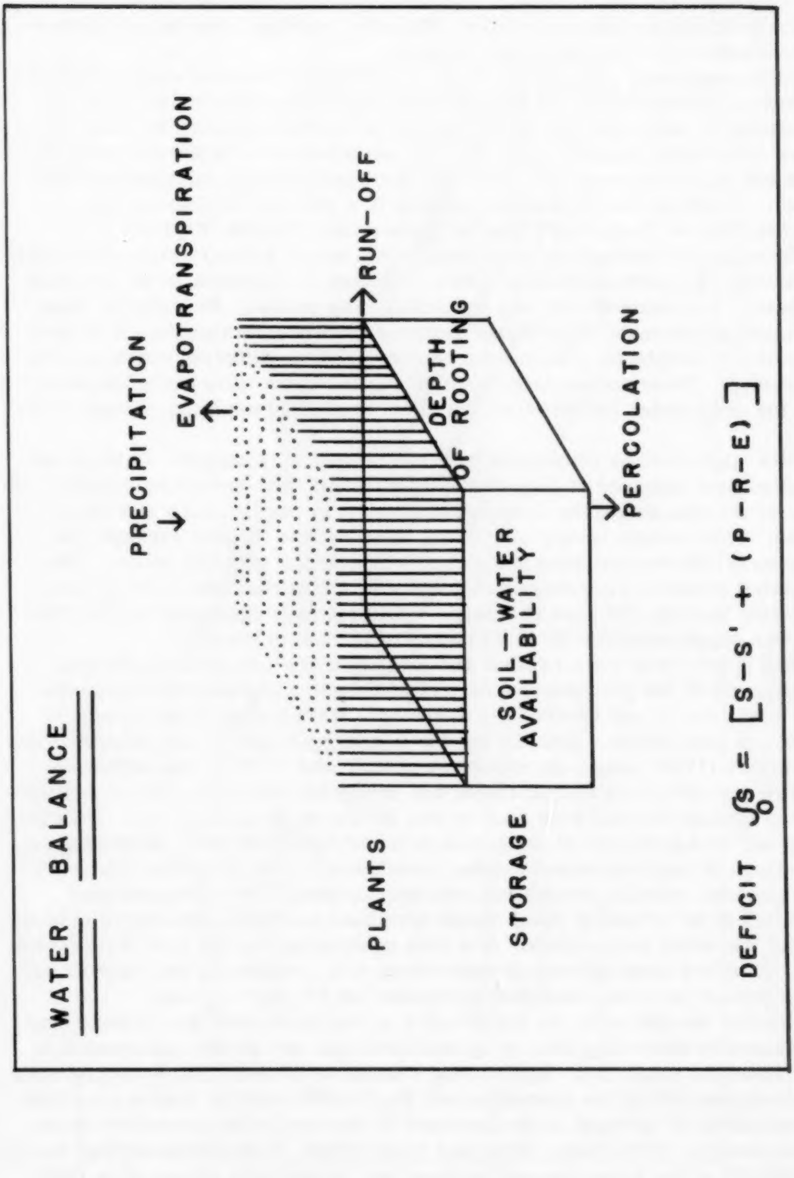

FIGURE I. DIAGRAMATIC WATER BALANCE

evapotranspiration depletes it. After water storage reaches its maximum value, additional precipitation is lost through deep percolation and run-off. Run-off is first subtracted from precipitation. It is interesting to note that even if the computed rate of evapotranspiration is in error by some considerable amount, the fluctuation of plant-soil water storage between fixed limits provides an auto-correction. Thus the resultant crop water balance determinations are not seriously affected.

Daily crop water balance can then be calculated from the meteorological records of precipitation and temperature. Agrohydrologic balance can be contrasted in unusually wet or dry years, or comparisons can be made between crop-water balance when specific combinations of different textured soils and different crops are involved. Such preliminary calculations were made to illustrate the crop water balance in a wet year at Gainesville, Florida (Figure 2) and a dry year at Gainesville, Florida (Figure 3).

These agrohydrologic balance parameters are of primary concern to agriculturists. Evapotranspiration deficit is of use to engineers in determining irrigation requirements for any particular crop season. Percolation value furnishes an estimate of fertilizer leaching tendency. Water deficit or percolation for months or seasons can be compared, or different years may be contrasted. These comparisons furnish a much more valid basis for assessing the crop-water balance than the more commonly used one, simple rainfall.

This might best be illustrated by taking a specific example. Using climatological data gathered at Gainesville by McCloud, and presented in table 1, it is observed that during the first 9 months of 1956 precipitation was 42.21 inches. This amount is only .05 inches less than the 51 year average, yet there were 103 drought days and a crop water deficit of 17.60 inches. The estimated potential evapotranspiration during the period was 53.96 inches indicating that the 103 days of drought would not have occurred had the rainfall been supplemented with 11.75 inches of irrigation water.

This is only true when rainfall and irrigation are 100 percent efficient, that is, none of the precipitation or irrigation water percolated through the plant root zone or can be credited to run-off. This degree of efficiency, of course, is improbable. Actually for the 9-month period, it was estimated that 5.85 inches (17.60 inches of crop water deficit less 11.75 inches difference between potential evapotranspiration and precipitation) or precipitation either leached through the soil root zone or ran off the surface of the soil. Efficiency during the application of irrigation water is highly variable, depending on the method of application and weather conditions. If an irrigation efficiency of 70 percent, which is considered average, is used, it is estimated that 25.20 inches of irrigation water would have been needed in addition to rainfall to meet the water requirements of a crop growing during the first 9 months of 1956. This is a large amount of water when it is considered that rainfall during the period involved, very nearly equaled the 51-year average.

A further insight as to the significance of the agrohydrologic balance may be obtained by observing data on agricultural land use as they influence data on agricultural water use. Again using Florida as an example, it may be seen that about one-half of the approximately 35,000,000 acres of land is farmland. Approximately 25 percent of the farmland is devoted to the production of improved pasture, field crops, citrus and truck crops. It is estimated that more than 820,000 of the acres planted to these four crops were irrigated in 1956. According to 1954 census figures approximately 428,000 acres of crops were

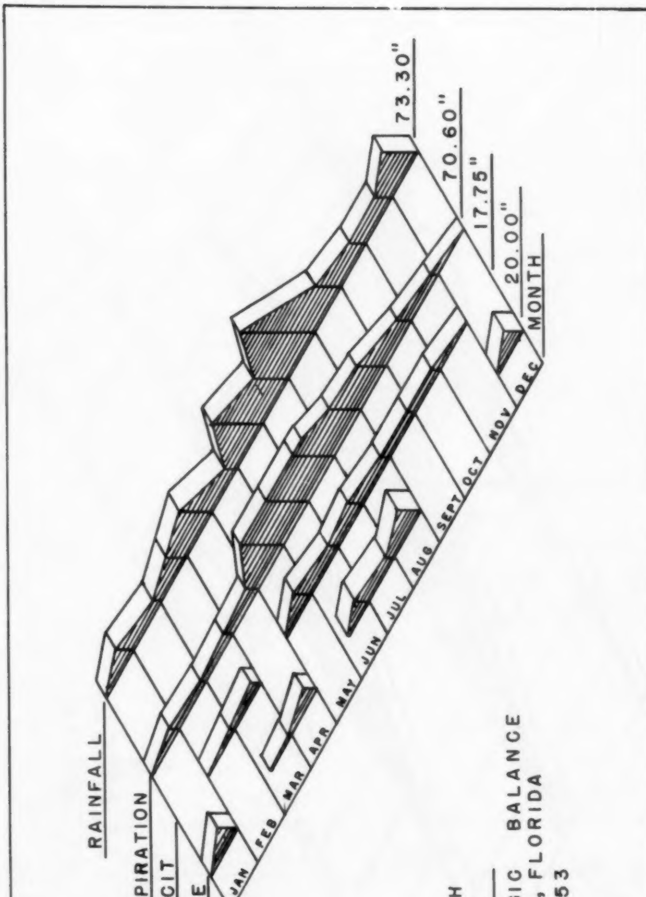

FIG. 2. AGROHYDROLOGIC BALANCE WITH TWO INCH STORAGE.
GAINESVILLE, FLORIDA, 1953.

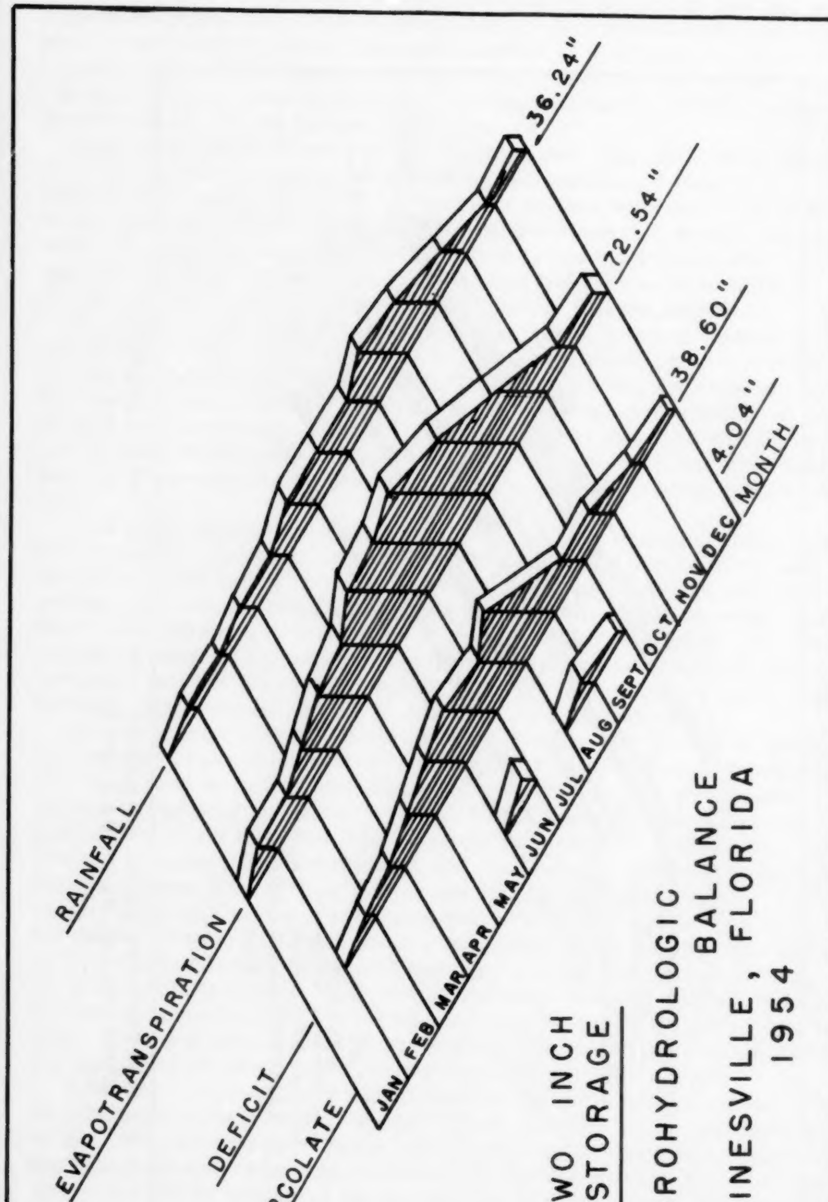

FIG. 3. AGROHYDROLOGIC BALANCE WITH TWO INCH STORAGE. GAINESVILLE, FLORIDA, 1954

Table 1

CLIMATOLOGIC DATA
Gainesville, Florida

Time	Precipitation	Estimated Potential Evapotranspiration*	Drought Days	Crop-Water Deficit**
Jan 1 - Sept. 30, 1955	38.97	60.15	122	26.13
Jan 1 - Sept. 30, 1956	42.21	53.96	103	17.60
Jan 1 - Sept. 30, (51 Yr.Av.)	42.26	- - -	- -	- - -

*Based on McCloud's evapotranspiration formula.
**Deficit based on a two inch available water storage, using the daily water budget and McCloud's evapotranspiration formula.

Table 2

CROP ACREAGE AND IRRIGATED ACREAGE FOR CERTAIN CROPS GROWN IN FLORIDA 1954-56

(All Figures Except Per Cent are In 1000 Acres)

Crop	Total Acres 1954 (1)	Acres Irrigated 1956 (2)	Per Cent Irrigated
Improved Pasture	1923	210	9.2
Field*	1470	- -	- -
(a) Tobacco	30	9	33.3
Citrus	565	242	42.8
Vegetables and Truck	321	273	85.0
Others**	- -	86	- -

(1) Source: 1954 Census of Agriculture (Preliminary).
(2) Source: Survey of county agents made by T. C. Skinner (6), Florida Agricultural Extension Service 1956.

*Tobacco Acreage Included - Insignificant acreage other than tobacco irrigated.
**Includes small acreages of Field Crops, nursery and greenhouse products, flowers, etc.

(6) Skinner, Thomas Co., Irrigation Survey for the State of Florida - 1956.

irrigated in 1954. This represents a sizable increase in irrigated acreage for a two year period during which it is estimated that only a slight increase was made in the acreage planted to these crops. Table 2 shows that irrigated acreages increased from 10.0 percent in 1954 to 19.2 percent in 1956.

Myers[7] has taken various climatological data together with available crop land-use and crop water-use data and related them as indicated in tables 3, 4, 5, 6, and 7. Table 3 shows the monthly median rainfall 1931-1936 for various sections of Florida. The state was divided into these six sections to add precision to the estimates.

Table 4 shows the estimated monthly evapotranspiration based on net solar energy. It was assumed that the potential evapotranspiration at Gainesville was representative for Northwest Florida, the potential evapotranspiration in Central Florida was representative for North Central and South Central Florida; and that potential evapotranspiration for Miami was representative for Southwest and the lower East Coast of Florida. From these two tables a comparison of potential evapotranspiration and the median rainfall for any of these given acres can be made.

Irrigated crop acreages were divided into five groups: (1) citrus, (2) vegetables and truck, (3) tobacco, (4) pastures and (5)"others" and growing seasons were established for each crop in each of the six sections of the state, and are tabulated in table 5.

A rainfall efficiency of 90 percent was used for citrus because the extensive root system of this crop reduces the likelihood of soil moisture percolating to a depth below the root zone and the nature of the soil on which most citrus is grown is not conclusive to run-off. For all other crops, 80 percent efficiency was used for months with median rainfall of less than 4 inches and 70 percent for months with a larger median rainfall. This group of crops are shallower rooted than citrus, thereby increasing the possibility of water loss by deep percolation. The lower efficiency allowed for months with heavy rainfall is due to the increased possibility, of run-off and deep percolation.

Computations, based on the procedures described in the above paragraphs and in consideration of data in tables 3, 4, and 5, give the estimated depths of supplemental irrigation water needed for crops in various sections of the State. These estimates are tabulated in table 6.

The present annual water requirement for agricultural crops is estimated to be more than 15 million acre feet. Of this amount, less than 1.5 percent is supplied to the crops by supplemental irrigation. When the increased economic benefits to agriculture that can be attributed to irrigation are weighed against the relatively small amount of water involved, there is little doubt as to the economic feasibility of irrigation in agriculture. The increasing rate to which this farming practice is put into operation on Florida farms justifies the predictions of many agricultural leaders that irrigation is still in its infancy in Florida as well as other humid areas of the country.

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7. Myers, J. M., Associate Agricultural Engineer, Fla. Agricultural Experiment Station.

Table 3

MONTHLY MEDIAN RAINFALL 1931-1956 FOR VARIOUS SECTIONS OF FLORIDA (1)

Month	Median Rainfall In Inches By Sections					
	North West (2)	North (3)	North Central (4)	South Central (5)	South West (6)	Lower East Coast (7)
January	3.33	2.61	1.32	1.37	1.00	1.81
February	3.45	2.58	2.46	1.88	1.58	1.82
March	5.06	2.99	3.22	3.08	2.14	2.54
April	4.34	2.68	3.16	2.92	2.97	3.62
May	3.69	2.58	3.41	3.21	3.74	3.94
June	5.44	6.05	7.22	7.06	7.96	6.78
July	7.79	7.71	8.42	8.05	8.28	6.44
August	6.54	6.56	7.22	6.76	7.12	6.69
September	5.37	6.88	6.40	6.72	7.73	7.58
October	1.52	3.80	3.76	3.34	3.82	8.32
November	2.44	1.46	1.45	1.38	1.30	2.27
December	3.71	2.36	1.80	1.63	1.22	1.50

(Table 3 cont'd)

(1) Information furnished by Mr. Keith Butson (8), State Climatologist, Florida Weather Bureau.

(2) Counties - Gay, Calhoun, Escambia, Franklin, Gadsden, Gulf, Holmes, Jackson, Jefferson, Leon, Liberty, Okaloosa, Santa Rosa, Wakulla, Walton, and Washington.

(3) Counties - Alachua, Baker, Bradford, Clay, Columbia, Dixie, Duval, Flagler, Gilchrist, Hamilton, Lafayette, Levy, Madison, Nassau, Putnam, St. Johns, Suwannee, Taylor and Union.

(4) Counties - Citrus, Hernando, Lake, Marion, Orange, Pasco, Seminole, Sumter and Volusia.

(5) Counties - Brevard, Desoto, Hardee, Highlands, Hillsboro, Indian River, Manatee, Okeechobee, Osceola, Pinellas, Polk, St. Lucie and Sarasota.

(6) Counties - Charlotte, Collier, Glades, Hendry, Lee, Martin, Monroe and Palm Beach.

(7) Counties - Broward and Dade.

(8) Butson, Keith, Unpublished Data, Florida Weather Bureau.

Table 4

ESTIMATED MONTHLY EVAPOTRANSPIRATION BASED ON NET SOLAR ENERGY*

Month	Potential Evapotranspiration Inches of Water (a) Gainesville	(b) Central Florida**	(a) Miami
January	2.98	3.63	4.27
February	3.47	3.82	4.17
March	4.94	5.64	6.34
April	6.13	6.25	6.83
May	6.17	6.39	6.61
June	6.77	6.22	5.67
July	6.07	5.85	5.63
August	6.23	6.10	5.97
September	5.74	5.62	5.50
October	4.74	4.81	4.88
November	3.49	3.83	4.17
December	2.64	3.04	3.43

*Data furnished by Dr. D. E. McCloud, Associate Agronomist, Agricultural Experiment Station.
**Average of Potential evaporization at Gainesville and Miami.

Table 5

LENGTH AND TIME OF CROP GROWING SEASON BY SECTION

Section of Florida

Crop	NW	North	N. Central	So. Central	SW	Lower East Coast
Citrus	x x x x x	x x x x x	12 Months	x x x x x	x x x x x	x x x x x
Vegetables and Truck	3 Months March-May	3 Months March-May	3 Months Feb.-April	3 Months Feb.-April	3 Months Jan.-March	3 Months Jan.-March
Tobacco	3 Months April-June	3 Months April-June	x x x x x x x x x x	x x x x x x x x x x	x x x x x x x x x x	x x x x x x x x x x
Pasture	8 Months Feb.-Sept.	8 Months Feb.-Sept.	10 Months Jan.-Oct.	10 Months Jan.-Oct.	12 Months	12 Months
Others*	x x x x x	x x x x x	x x x x x	x x x x x	x x x x x	x x x x x

*Growing Season Variable

Table 6

ESTIMATED ANNUAL DEPTH OF SUPPLEMENTAL IRRIGATION WATER NEEDED FOR AGRICULTURAL CROPS

(All Values Are In Inches)

Crop	NW	North	N. Central	So. Central	SW	Lower East Coast
Citrus			13.4			
Vegetable & Truck	8.4	11.4	9.2	10.1	11.8	10.6
Tobacco	11.4	11.4				
Pasture	16.7	19.1	21.5	23.2	28.0	25.6
Other	12.2	12.2	12.2	12.2	12.2	12.2

Table 7

Table 7

ESTIMATED SUPPLEMENTAL IRRIGATION WATER

REQUIREMENT FOR CROPS IN FLORIDA - 1956

Crop	Acre Feet (Thousands Per Year)
Citrus	261
Vegetables and Truck	232
Tobacco	9
Pasture	453
Other	87
Total	1042

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Proceedings of the American Society of Civil Engineers

THE SOAP LAKE BASIN[a]

Keith E. Anderson[1]
(Proc. Paper 1384)

INTRODUCTION

The somewhat unique problems associated with the interception, control, or removal of water from the Soap Lake area are of considerable interest from the standpoint of irrigation and drainage engineering. This paper will discuss only the technical engineering or scientific aspects of the problem, with only brief mention of the admittedly controversial matters that lie more properly within the field of the administrator, public relations specialist, attorney, and medical practitioner.

Soap Lake is a relatively small natural lake located about six miles northeast of Ephrata, Washington near the northern limit of the irrigated lands of the Columbia Basin Project in central Washington. It has a surface area of about 840 acres at its present water-surface elevation of 1076 and contains approximately 25,000 acre-feet of mineralized water. It lies in a closed basin and is the southernmost member of a chain of lakes in the lower Grand Coulee. The lake has no natural outlet and loses water only by evaporation. The water consequently is very highly mineralized as a result of countless years of concentration. The lake derives its name from the suds-like foam piled up on the shore on some windy days.

The lake is fed primarily from subsurface sources with water entering through basalt bedrock or gravel-filled channels. The area of the watershed tributary to the lake is approximately 33 square miles; the surface runoff from nearly 14 square miles of this area is largely intercepted by the main West Canal. The area from which ground water is tributary to the lake is doubtless much larger than the surface drainage watershed.

Over a period of many years, the mineralized lake waters have developed a "layered" condition—with an upper layer of less mineralized water floating upon a lower, denser, more highly mineralized water. The two layers do not mix, even during normal cycles of temperature changes. The upper layer is alleged to be of therapeutic value and is presently used for bathing and internal use.

Prior to 1947 the upper layer of water, comprising approximately 24,000

Note: Discussion open until February 1, 1958. Paper 1384 is part of the copyrighted Journal of the Irrigation and Drainage Division of the American Society of Civil Engineers, Vol. 83, No. IR 2, September, 1957.

a. Presented at a meeting of the ASCE in Spokane, Wash., September, 1956.
1. Regional Drainage Engr. Bureau of Reclamation, Boise, Idaho.

acre-feet and extending to a depth of about 62 feet, had a salinity of about 35,000 to 40,000 parts per million (3.5 to 4.0 percent). The lower layer of water, comprising approximately 1,000 acre-feet and extending to the maximum lake depth of 96 feet, has a salinity ranging from 40,000 to 144,000 parts per million (4.0 to 14.4 percent). The mineral concentration of the surface waters of the lake have diminished from nearly 40,000 parts per million prior to 1947 to less than 25,000 parts per million in 1955. This dilution resulted from the increased inflow to the lake in that period and the pumping from the lake necessary to prevent flood damage. Pumping from the lake in 1953-55 removed an estimated 374,000 tons of dissolved solids, with some 800,000 tons still remaining in the upper "layer."

In the decade prior to 1947 the lake level remained fairly constant because of the natural balance between inflow and evaporation. In recent years, however, the lake level has been rising because of increased precipitation, subnormal evaporation, and the effects of irrigation operations on the Columbia Basin Project. The West Canal, a major feature of the project, traverses on the east, north, and west of the lake and furnishes irrigation water to some 2,400 acres of land in the Soap Lake Basin.

The City of Soap Lake, a community of about 2,500 people, lies on the south shore of Soap Lake. The rising lake level has endangered the city sewage disposal system, has flooded beaches and some buildings, and has contributed to flooding of basements adjacent to the lake. The primary State highway paralleling the east shore of the lake has also been endangered. The problem has then been, among other things, to establish the sources and amounts of inflow to the lake, to determine how the excess inflow can be intercepted or removed to prevent flood damage, and—if possible—to prevent excessive dilution of the mineralized lake water.

Geology and Physiography

The east and west sides of Soap Lake are bounded by high basalt cliffs which rise several hundred feet above the lake. These cliffs are the sides of the lower Grand Coulee, an ancient temporary channel of the Columbia River. Lake Lenore, a slightly larger and less mineralized body of water, lies to the north of Soap Lake. The two lakes are separated by a low divide of gravel and boulders which rise 30 or more feet above Soap Lake. To the south, the basin is blocked by a fill of sand, gravel, and boulders and some basalt. The lowest point on this divide to the south is about 80 feet above Soap Lake.

During recent geologic time, the Columbia River was blocked downstream from the present Grand Coulee Dam, Washington, by a huge mass of ice that squeezed down from the north. This created a huge lake that backed into Canada and Idaho, and eventually spilled over its side and flowed south into the Quincy basin, creating another lake, which, in turn, overflowed south and returned to the Columbia River. This flood of river and glacier melt-water created an immense waterfall a few miles north of what is now Coulee City, Washington. The face of the falls was undermined and receded northward to the Columbia River—thus creating the upper Grand Coulee.

A waterfall also developed at what is now Soap Lake and receded to the present Dry Falls at the north end of the lower Grand Coulee. About forty cubic miles of rock were ripped out of the two coulees and some of it was deposited in the Quincy basin as silts, sands, gravels, and boulders. Soap

Lake lies in the lowest portion of the lower Grand Coulee, with the southern end blocked by the debris from the coulee excavations and by some firm rock.

Lake Levels

Lake levels in the northwest have been rising for the past several years. While the exact figures are not available, it is known that Medical Lake, near Spokane, Washington, has been rising quite steadily and is now 20 feet higher than in 1893. Jameson Lake, which is in Moses Coulee and about 25 miles northwest of Ephrata, Washington, has also been rising. This reflects a climatic condition of above normal precipitation, so that lakes with no outlets logically should be rising. The start of irrigation operations on the Columbia Basin Project coincided with this high climatological cycle and accompanying higher lake levels in the area.

The principal inflow to Soap Lake is ground water entering the lake through basalt flows and gravel-filled channels. With no natural outlet, concentration by evaporation has resulted in its becoming extremely mineralized and alkaline. The dissolved mineral content of surface waters of the lake was about 24,500 parts per million in October 1955. The natural conditions of evaporation and precipitation for many years have been such as to maintain rather well the delicate balance necessary to prevent either complete evaporation of the lake, or a steady rise of water that would inundate the entire basin. While this over-all balance has been maintained in the past, there have been many cyclical fluctuations in the lake level. According to statements made by early residents, the lake elevation was higher than it is at present, in 1900, 1903, and 1916 and probably reached an elevation exceeding 1084. The highest apparent water line on rocks bordering the lake, as indicated by alkali deposits, is at elevation 1083.1.

Between 1938 and early 1948, the lake level remained fairly constant, with the seasonal fluctuation varying between elevations 1071 and 1073; about four feet below the present water level. From 1948 to 1951, the Soap Lake level rose from elevation 1073 to elevation 1077; this rise is attributed entirely to the above-normal precipitation trend during this period. At the beginning of the irrigation system testing in September 1951, the water surface elevation of Soap Lake was approximately 1076.3, after being lowered by evaporation about 0.7 from the seasonal high in May. A study of records indicates a sharp rise of inflow late in 1952, which is logically attributed to a new source or sources of water resulting from irrigation operations in the Columbia Basin Project. After pumping from the lake and interception wells during 1953, 1954, and 1955, the lake level on January 1, 1956 was 1076.4.

Analysis of Inflow

By the close of 1952 the level of Soap Lake was rising rapidly, even when allowance was made for the wet precipitation cycle. A detailed analysis was then started to determine—if possible—the normal inflow to the lake prior to irrigation development, the variations from normal due to climatological trends, and the inflow resulting from irrigation. Several methods of analysis have been used by different investigators but the procedure described here was based on evaporation-precipitation data.

Fairly complete summer evaporation records were available from a

Class A Weather Bureau pan near Quincy, some 20 miles southwest of Soap Lake. Missing data were estimated using correlations with the Prosser, Washington station. Winter evaporation (November-March) was assumed as 15 percent of the annual total. Monthly Soap Lake evaporation was then tabulated, using 70 percent of pan evaporation for the lake surface. These computed monthly evaporations are the losses from the lake. Using available data on lake level fluctuations to indicate changes in lake storage, monthly inflows in terms of feet of depth on the lake were readily computed. Knowing the lake surface area, these inflows were converted to acre-feet values and monthly normals determined for the 1941-51 base period.

Two curves were then plotted showing monthly cumulative departures from normal—one for precipitation at Ephrata, the closest rainfall station, and the other for lake inflow computed as just described. By adjusting the vertical scales on these curves, a marked degree of parallelism could be shown between precipitation and inflow—until late 1952 when the inflow increased abruptly as a result of irrigation operations. Projecting the inflow curve parallel to the precipitation curve permitted estimating the inflow attributable solely to irrigation developments.

To illustrate, the inflows in acre-feet for 1954, computed in the foregoing manner, are as follows:

Month	Normal (1941-51)	From Natural Sources	From Irrigation	Total
Jan.	310	310	680	990
Feb.	340	340	490	830
Mar.	330	280	560	840
Apr.	370	300	390	690
May	410	310	400	710
June	410	380	260	640
July	380	370	280	650
Aug.	290	280	440	720
Sept.	240	230	550	780
Oct.	230	220	540	760
Nov.	320	260	650	910
Dec.	340	260	670	930
Year	3,970	3,540	5,910	9,450

An independent analysis of inflow resulting from irrigation was made, based on estimates of return flow from known irrigated areas, seepage losses from canals and laterals, and increases in ground-water storage. This analysis resulted in an estimate of 6,600 acre-feet for 1954—agreeing fairly well with the 5,910 acre-feet from the evaporation-precipitation study.

Lake Level Control

Knowing the amount of added inflow resulting from irrigation operations, the next problem is that of removing this excess to prevent a steady rise of lake level. This removal is complicated by several factors: if water is pumped directly from the lake it results in "freshening" of the mineralized water which is opposed by many local residents; undiluted lake water cannot be safely carried in the concrete-lined West Canal or the Soap Lake Siphon;

dilution water must come from the primary irrigation system starting at Grand Coulee Dam with a lift of nearly 300 feet; to avoid contaminating irrigation supplies, any mineralized water removed—even if diluted—must be lifted some 250 feet into the West Canal, must be removed in winter months, and must be carried some 35 miles to a wasteway to the Columbia River.

The plan developed and now operating has been to intercept as much inflow as readily possible before it reached the lake, with supplemental pumping as needed directly from the lake. Geological and geophysical exploration revealed several channels in bedrock, filled with permeable sediments, at each end of the lake. Interception wells have been drilled which are pumped continuously, discharging the fresh-water inflow into the West Canal. A temporary pumping plant in the lake operates during winter months, discharging the mineralized water into the West Canal system where it is diluted with Columbia River water.

Excess inflow to Lake Lenore, north of Soap Lake, is also pumped in winter months and diluted in similar fashion. By mid-1958 the level of Lake Lenore should be lowered to or below the Soap Lake level which will stop the small underflow presently migrating southward into Soap Lake.

Pumping operations by the Bureau of Reclamation during 1953-55 resulted in the removal of some 44,000 acre-feet of water. About one-half of this was pumped from Lake Lenore; the other half from the Soap Lake basin—about 1/2 from wells and 1/2 directly from the lake.

If no pumping had been done, the level of Soap Lake at the end of 1955 would have stood some 25 feet above its present level—inundating most of the City of Soap Lake and the highway bordering the lake.

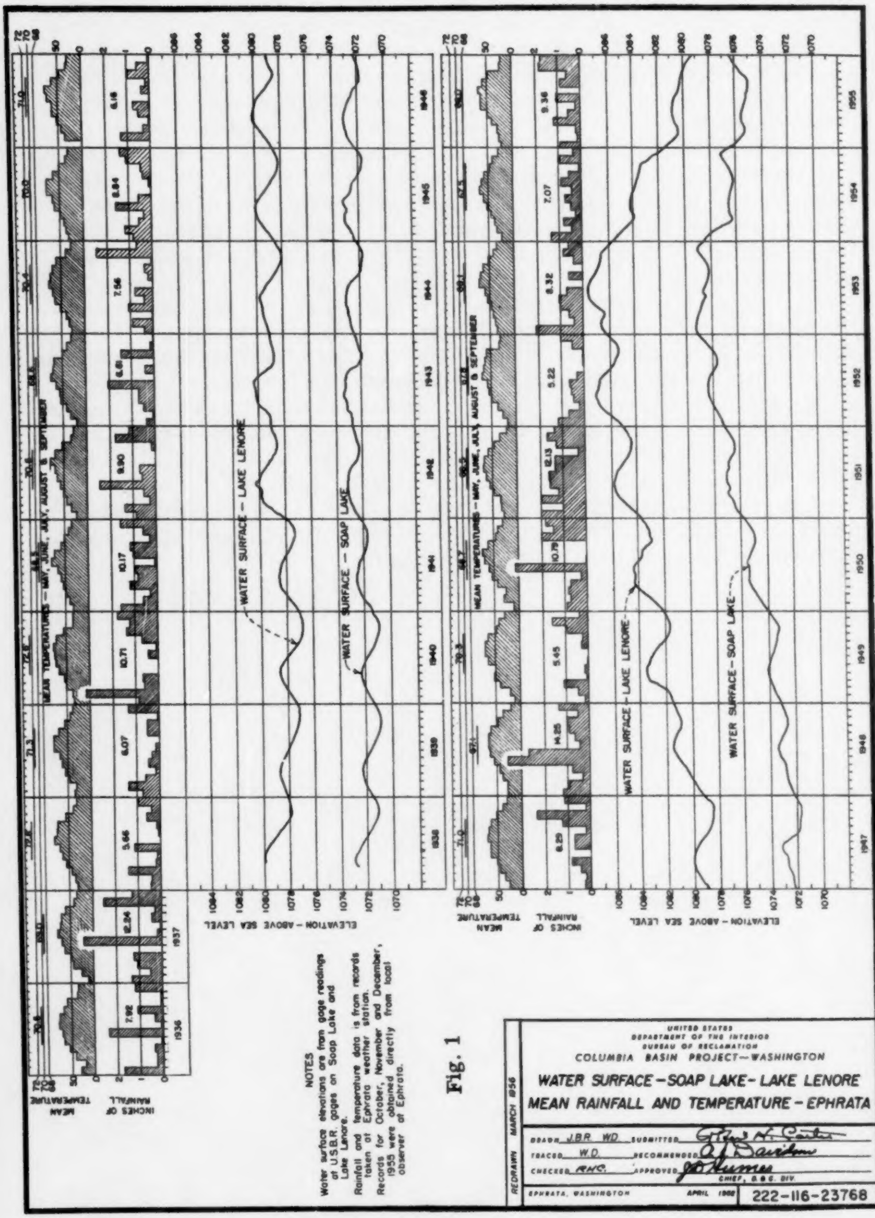
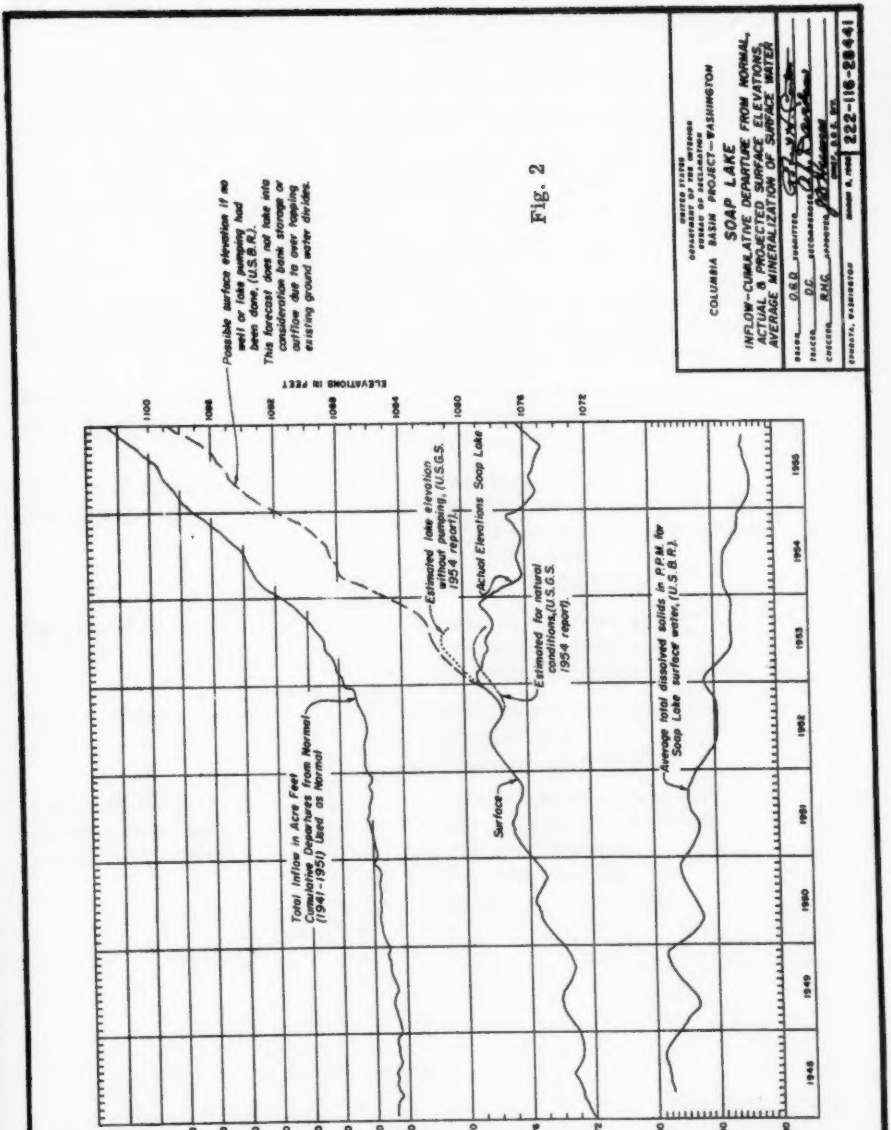

MEAN TEMPERATURE
INCHES OF RAINFALL
ELEVATION - ABOVE SEA LEVEL
MEAN TEMPERATURES - MAY, JUNE, JULY, AUGUST & SEPTEMBER
WATER SURFACE - LAKE LENORE
WATER SURFACE - SOAP LAKE
NOTES
Water surface elevations are from gage readings of U.S.B.R. gages on Soap Lake and Lake Lenore.
Rainfall and temperature data is from records taken at Ephrata weather station.
Records for October, November and December, 1955 were obtained directly from local observer of Ephrata.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT—WASHINGTON
WATER SURFACE—SOAP LAKE—LAKE LENORE
MEAN RAINFALL AND TEMPERATURE—EPHRATA
REDRAWN MARCH 1956
EPHRATA, WASHINGTON
222-116-23768

Fig. 1



UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT—WASHINGTON
SOAP LAKE
INFLOW—CUMULATIVE DEPARTURE FROM NORMAL, ACTUAL & PROJECTED SURFACE ELEVATIONS, AVERAGE MINERALIZATION OF SURFACE WATER
222-116-28441
ACRE FEET
ELEVATIONS IN FEET
PARTS PER MILLION
Total Inflow in Acre Feet Cumulative Departures from Normal (1941-1951) Used as Normal
Surface
Estimated for natural conditions,(U.S.G.S. 1954 report).
Estimated lake elevation without pumping, (U.S.G.S. 1954 report).
Actual Elevations Soap Lake
Possible surface elevation if no well or lake pumping had been done, (U.S.B.R.). This forecast does not take into consideration bank storage or outflow due to over topping existing ground water divides.
Average total dissolved solids in P.P.M. for Soap Lake surface water, (U.S.B.R.).

Fig. 2

DIVISION ACTIVITIES
IRRIGATION AND DRAINAGE DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

September, 1956 - September, 1957

Harry F. Blaney (Chairman)	U. S. Department of Agriculture 1505 Post Office-Court House Los Angeles 12, California
Howard T. Critchlow (Vice-Chairman)	Consulting Engineer 577 Rutherford Avenue Trenton, N. J.
William W. Donnan (Secretary)	Area Supervisor Soil & Water Conservation Div. U. S. Department of Agriculture P. O. Box 672 Riverside, California
Hjalmar T. Person	Dean of Engineering University of Wyoming Laramie, Wyoming
John H. Bliss	Interstate Stream Engineer Capitol Bldg. Santa Fe, New Mexico
Finley B. Laverty (Contact Member - Board of Direction)	Chief Hydraulic Engineer Los Angeles County Flood Control District 502 Lakeview Rd. Pasadena, California

Review of Division Activities

The Irrigation and Drainage Division has held several successful Division Conferences and Technical Sessions during the past year.

On September 6, 7, and 8, 1956 a Division Conference was held at the Davenport Hotel in Spokane, Washington. The Spokane Conference was one of the best meetings ever held by the Division. Thirteen papers were presented on irrigation, drainage and use of water. About 100 members and guests registered for the conference and a large number made the field trip to Coulee Dam on September 8.

Three Technical Sessions were sponsored by the Division Committee on Irrigation and Drainage Practices in Humid Areas during the Society Convention held in Jackson, Mississippi the week of February 18-22, 1957. One session was devoted to papers on sprinkler irrigation, one session on gravity irrigation, and one session on drainage.

On April 29 and 30, 1957 a Division Conference was held at the Sheraton-Palace Hotel in San Francisco, California. This meeting was a part of an Intersociety Conference on Irrigation and Drainage sponsored by the Irrigation and Drainage Division of the American Society of Civil Engineers in conjunction with the Soil and Water Division of American Society of Agricultural Engineers and the Soil Conservation, Irrigation, Drainage, and Soil Tillage Division of Soil Science Society of America. The 370 registrations for this conference exceeded all expectations. The theme of the meeting was, "Can Man Develop a Permanent Irrigation Agriculture?" The program Committee included Harry F. Blaney (ASCE, ASAE), Robert M. Hagan (ASAE, SSSA), N. R. Lewis (ASCE, ASAE), Walter W. Weir (ASCE, ASAE, SSSA), Frank B. Glendenen (ASCE), and Bert L. Smith (Editor, Western Water News). A total of 28 papers were presented at this conference, 10 of which were given by members of the ASCE. About 200 in attendance were from foreign countries since the Intersociety conference preceded and was held in conjunction with the Third World Congress of the International Irrigation and Drainage Commission. Over 450 representatives from 41 foreign countries and the United States participated in this Congress the first week in May. Following the San Francisco meeting a group of about 150 made a two-week tour of California water and irrigation projects and held a final meeting in Los Angeles, May 18, 1957.

Executive Committee, Irrigation and Drainage Division

The Executive Committee was held three meetings during the past year.

On September 7, 1956 a meeting was held at Spokane, Washington at the time of the Spokane Conference. This was the last meeting under the charimanship of George N. Carter. Some of the highlights of that meeting were as follows:

a) Herbert Prater, who had served faithfully as Secretary to the Executive Committee for 5 years, resigned. William W. Donnan was elected to the position replacing Prater.

b) Harry F. Blaney was elected Chairman of the Executive Committee for the year beginning October 1, 1956.

c) Howard Critchlow was elected Vice-Chariman.

The next meeting of the Executive Committee was held in Salt Lake City, Utah on November 14, 1957. All members were present including Finley

Laverty, the Board of Direction Contact Member, and Don Reynolds, Assistant to the Secretary, ASCE.

The main items covered in this meeting were the planning for the San Francisco Conference and the final plans for the technical sessions at the Jackson, Mississippi Convention.

On April 12 and 13, 1957, the Chairman, Vice-Chairman and Secretary of the Executive Committee attended the Societies' annual Technical Procedure Conference which was held in Oklahoma City, Oklahoma. These officers spent two days participating in a full schedule of reports, discussions, and exchange of information designed to expand the effectiveness and extent of constructive activities of the various Technical Divisions.

On April 28, 1957 the Executive Committee met in the Sheraton-Palace Hotel, in San Francisco. At this meeting a complete review was made of all the activities of the various technical committees and their reorganization was discussed. The Society was petitioned to provide for rotation of membership on the Executive Committee in line with Handbook Procedure. The Budget for the Division for the coming fiscal year beginning October 1, 1957 was reviewed and adopted. It was decided that the Division would sponsor four technical sessions at the Society Conventions to be held in Chicago in February 1958, and four sessions at the Portland, Oregon Convention in June 1958. In closing, the Executive Committee elected Howard T. Critchlow as Chairman, Hjalmar T. Person as Vice-Chairman, and William W. Donnan as Secretary for the year beginning October 1, 1957.

ADMINISTRATIVE COMMITTEE ACTIVITIES

Committee on Publications

Purpose: To review critically and edit papers submitted for publication in the Society Publications.

Membership: Carl Wilder, Chairman, Charles L. Barker, Dean F. Peterson, Jr., and Calvin C. Warnick.

Accomplishments: This Committee has reviewed many papers during the past year and has recommended some of them for publication. A back-log of 12 papers are now in the hands of the Committee. All business has been handled by correspondence. This Committee has done excellent work under the leadership of Chairman Wilder during the past two years, and his resignation effective October 1, 1957 is regretted. The Executive Committee has appointed Dean F. Peterson, Jr. as new Chairman, and Harry F. Blaney as a member of the Committee on Publications, effective October 1, 1957.

New Activities Planned: This Committee will review papers submitted for publication throughout the coming year.

Committee on Cooperation with Local Sections

Purpose: To administer procedures which will establish constructive activities at local section level in the field of interest of this division, and which will provide support for activities of the division, subject to review of procedures by the Society.

Membership: S. Mark Davidson, Chairman, Rowland W. Fife, Arthur D. Soderberg.

Accomplishments: This new Committee has had no activity during the year.

New Activities Planned: The Committee will meet this summer of 1957 and will undertake its duties.

Committee on Session Programs

Purpose: To administer the preparation of programs for presentation under sponsorship of the division and the conduct of such programs in cooperation with convention technical program chairmen.

Members: John H. Bliss, Chairman, George D. Clyde, O. W. Israelsen, Herbert E. Prater, Kenneth Q. Volk.

Accomplishments: This committee arranged the programs for three technical sessions at the Jackson, Mississippi Convention in February 1957, and for the Joint Intersociety Conference held in San Francisco in April 1957.

New Activities Planned: Programs will be arranged for the technical sessions to be sponsored by the Division for the coming Chicago, Illinois and Portland, Oregon conventions.

TECHNICAL COMMITTEE ACTIVITIES

Committee on Consolidation and Progressive Betterment of Old Irrigation Systems

Members: A. Alvin Bishop, Chairman, George D. Clyde, John E. Hayes, Dean F. Peterson, Jr., L. M. Winsor.

Accomplishments: This committee held one meeting in Salt Lake City and drafted an outline for a proposed report or manual on the problem. This outline was reviewed by the Executive Committee and some recommendations were made to condense the objectives.

New Activities Planned: One meeting is planned for the coming year to formulate final plans for a report.

Committee on Drainage and Irrigation Lands

Members: Charles R. Maierhofer, Chairman, William W. Donnan, Edwin E. Elliott, O. W. Israelsen, Ronald C. Reeve.

Accomplishments: One informal meeting was held in Spokane, Washington in September 1956. The Committee revised its objectives and resolved to complete various chapters of a proposed report on drainage.

New Activities Planned: Individual work assignments were made with a target date for completion of write-ups by May 1958. It was proposed that one session at the Portland Convention in June 1958 be devoted to drainage reports with this Committee acting as sponsor.

Committee on Ground Water

Members: Harvey O. Banks, Chairman, Frederick L. Hotes, Harris R. McDonald, Cleve H. Milligan, Dean C. Muckel, Harold C. Schwalen, Harold E. Thomas, Robert O. Thomas, Secretary.

Accomplishments: This Committee has completed six of the nine proposed chapters of the manual. These completed chapters are now being reviewed.

New Activities Planned: Upon completion of the write-ups for all chapters the Committee will hold one meeting for final review before typing. The Committee requested that money be budgeted for the cost of typing this manual.

Due to the resignation of Fred Hotes, Robert O. Thomas was recommended for replacement on the Control group of this Committee.

Committee on Irrigation and Drainage Practices in Humid Areas

Members: Paul H. Berg, Chairman, Marion C. Boyer, Fred H. Larson, Leo F. Reynolds, Harold A. Scott, Sr., Howard T. Critchlow (Executive Committee Contact Member).

Accomplishments: This Committee was organized in June 1956 and has held several meetings. It has sponsored technical sessions at Jackson, Mississippi and has been responsible for the development of the Task Committee on Water Rights Laws in the Humid Area States.

New Activities Planned: This Committee plans to have a meeting in Atlanta, Georgia in 1958 in conjunction with a Hydraulics Division Conference. It also proposes to develop several new task force committees on various phases of Humid Area problems.

Task Committee on Water Rights Laws in States in the Humid Area

Members: Paul Berg, John J. Ledbetter, James I. Seay, Jr., George R. Shanklin, David B. Smith, Robert L. Smith.

Accomplishments: This Committee met for the first time in August 1956 and proposes to study the problem of Water Rights Laws. Assignments were made for drafting certain statements and preparing reports on the problem. Some members of this Committee met in Jackson, Mississippi in February 1957.

New Activities Planned: This Committee plans to hold one meeting at a central location during the late fall or winter of 1957.

Committee on Research

Members: Gerald B. Keesee, Chairman, Maurice L. Albertson, Harry F. Blaney, Jr., Dell G. Shockley, Robert L. Hardman.

Accomplishments: This Committee was organized in May 1956 and has held two subsequent meetings. As a result of these meetings a comprehensive questionnaire has been prepared to assess the present state of and need for future research on irrigation and drainage problems. This questionnaire was reviewed by the Executive Committee.

New Activities Planned: A revised questionnaire will be sent out early this fall and the results will be analyzed. The Committee proposes to hold one formal meeting this coming year. The Executive Committee has decided to appoint one more member to the Research Committee, preferably from the Humid Area.

U. S. National Committee of the International Commission on Irrigation and Drainage

Members: Walter E. Blomgren, Chairman, Charles R. Maierhofer, Gerald T. McCarthy, C. J. McKnight, George O. Pratt, Harold A. Scott, Sr., Ivan D. Wood.

Accomplishments: This Committee, in cooperation with the Irrigation and Drainage Division of the San Francisco ASCE Section, sponsored the Third Congress of the International Commission on Irrigation and Drainage which was held in San Francisco April 29 to May 4, 1957. Over 450 representatives from 41 countries attended this Congress and presented 29 papers on canal lining, soil water relationship in irrigation, hydraulic structures, and interrelation between irrigation and drainage. These have been printed in seven volumes.

Announcements

The Executive Committee voted to fore-go having an Irrigation and Drainage Division Conference this coming year. This action was taken because of the Society Conventions scheduled for Chicago, Illinois and Portland, Oregon in 1958. There will be plans forthcoming on the proposed technical sessions for the Chicago Convention, February 24-28, 1958 at the Sherman Hotel, and for the Portland Convention to be held in June 1958.

Since these conventions will take the place of Division Conferences next year we are hoping to be able to sponsor an outstanding series of technical sessions. It is hoped that many of the Society members will make plans to attend either the Chicago Convention or the Portland Convention next year.

At the ASCE Summer Convention held in Buffalo, New York on June 3-6 the Society appointed Mr. Kenneth Volk, Consulting Engineer, Los Angeles, California to be the new member of the Executive Committee. The Executive Committee, which will take office on October 1, 1957 will be as follows:

	Term Expires	
Howard Critchlow	1958	Chairman
H. T. Person	1959	Vice-Chairman
John H. Bliss	1960	
Kenneth Volk	1961	
William W. Donnan		Secretary
Finley Laverty		Contact Member

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

VOLUME 82 (1956)

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068 (WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092 (HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103 (IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c], 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127 (SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

VOLUME 83 (1957)

JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158 (EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)[c], 1193 (PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203 (SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218 (SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226 (WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246 (HW2), 1247(HW2), 1248(WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257(IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267 (PO3), 1268(PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275 (SA3), 1276(SA3), 1277(HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283 (HY3)[c], 1284(PO3), 1285(PO3), 1286(PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297(HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303 (ST4), 1304(ST4), 1305(SU1), 1306(SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315(ST4), 1316(ST4), 1317(ST4), 1318 (ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324(AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338(SA4)[c], 1339(CO1) 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347(BD1), 1348(HY4)[c], 1349(SA4)[c], 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360(IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369(WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378(HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

c. Discussion of several papers, grouped by Divisions.